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**PROPERTIES
OF SOILS
AND THEIR
MEASUREMENT**

SECOND EDITION

Joseph E. Bowles

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ENGINEERING PROPERTIES OF SOILS AND THEIR MEASUREMENT

SECOND EDITION

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ENGINEERING PROPERTIES OF SOILS AND THEIR MEASUREMENT

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PREFACE

This edition has been rewritten and generally updated to reflect more recent testing procedures/modifications, user comments, and the transition of the United States to the SI system of units. I have added two simple introductory experiments (Nos. 21 and 22), and at the end rather than at the beginning because some of Experiment Nos. 1-20 are on TV tapes or have otherwise been permanently identified. Users may wish to employ Experiment No. 21 as a first experiment or as a first or second class meeting demonstration of volumetric-gravimetric relationships and as an introduction to the soils laboratory.

This edition is totally SI/metric units. I call the units SI/metric because both SI and "preferred usage" metric units are commonly used and/or found on equipment in soils laboratories. Further, in spite of an intensive push to pure SI, the centimeter, gram, and cubic centimeter will continue to be used as a matter of practical necessity.

A major change from the first edition is the binding of the data sheets with the text. This was necessitated by the tremendous cost of binding the data sheet package. The method of binding allows the student to tear out the data blank, then remove the serrated edge by tearing along the perforations, thus maintaining the text and producing a nearly standard-page-size data sheet suitable for reports. This method allows use of metric grids on the furnished graph sheets.

In several of the experiment/projects I have indicated practical methods and/or preliminary work which should be done prior to the laboratory exercise as well as equipment which can be easily fabricated to improve the project.

As in the first edition, this laboratory text has been written to introduce the basic essentials of the laboratory testing of soils, including methods of data collection, computations, and the presentation of results. The general format of equipment lists, photographs, typical data sheets, and data reduction has been retained from the first edition. All the illustrative data sheets have been redone with actual soil test data obtained by me, or under my supervision, using metric equipment and not merely conversion of units.

The tests in this manual follow rather closely the ASTM or AASHTO standard test procedures. Principal exceptions are the use of distilled water and/or highly specialized equipment (not likely to be found in student laboratories) and deviations from the time element which may be an essential feature of some of the standard test procedures. Any deviations are noted so that the user will be aware of the fact when the test procedure is nonstandard.

I would like to express especial appreciation to Professor E. Schaeffer of St. Lawrence College, Ontario, Canada, and to Professor Turgut Demirel of Iowa State University, who critically reviewed the entire manuscript. I would also like to convey my thanks to the many students (graduate and undergraduate) whose ideas were gleaned from their laboratory reports and used to improve the laboratory work/methodology.

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**ENGINEERING
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SOIL MECHANICS DEFINITIONS, LABORATORY PROCEDURES, AND REPORT PREPARATION

Introduction

This laboratory manual section provides information of a general reference nature. It contains a brief reference article on soil mechanics definitions as well as volumetric and gravimetric relationships. Also included in this section are information on laboratory procedures the student is expected to use and a guide to the preparation of laboratory reports which the student may find useful. The student should carefully read this section of the manual and refer to it often to ensure correct report form and for methods of preparing graphs to present test information which requires curve plotting.

SI/Metric Units Used in This Text

No foot-pound-second (fps) laboratory units are used in this text. Commonly, soils laboratory equipment will measure in the standard SI unit of *mass*, the kilogram (kg), or in the smaller (non-SI unit) mass unit of grams (g). Weight as used by engineers is a force unit (but is hardly ever stated as such). We have the following considerations:

$$F = ma = \frac{W}{g}(a)$$

and if we substitute the standard gravitational acceleration, g , for the acceleration in the above equation we have $F = W$ (where W is the body force or weight produced by the gravitational effect of the earth on the mass of the body under consideration). The standard gravitational acceleration (which varies slightly with location) is based on elevation of sea level at 45°N latitude and is very nearly 9.807 m/s². The standard SI force unit is the Newton (N) which consists of 100,000 dynes. The Newton is also the force which will accelerate a mass of 1 kg at a rate of 1 m/s². One gram force = 980.7 dynes (also the acceleration of gravity in cm/s²), and thus we have:

$$F = 1 \text{ gm (mass)} \times 980.7 \text{ cm/s}^2 \quad (\text{units of force} = \text{dynes})$$

If we divide by 980.7 dynes/gm, it is clear that grams mass and grams force are interchangeable (also kilograms) as long as we deal with the standard gravitational constant. This has caused considerable confusion in the past, but it may become less of a problem in the future since the Newton (force) uses an acceleration of 1 m/s² (not the acceleration of gravity).

For soils work we will use the following units (with standard abbreviations shown). Note that the intermediate units used will be consistent with the laboratory equipment, and the final units will be reported in SI.

Quantity	Nonstandard Intermediate Unit	SI Unit
Length	centimeter (cm)	millimeter (mm), or meter (m)
Volume	cubic centimeter [cm^3 , cu cm, or cc—also, milliliter (ml)]	meter ³ (m^3)
Mass	gram (g)	kilogram (kg)
Weight	g or kg	Newton (N) or kilonewton (kN)
Unit weight	g/cm^3 ; sometimes tonnes/ m^3 tonne = 1000 kg	kilonewton/ m^3 (kN/m^3) kN = 1000 N
Pressure	kg/cm^2	N/m^2 [Pascal (Pa); in soils work use kilopascal (kPa)]
Energy		Newton-meter = Joule (J) Moment = $\text{N} \cdot \text{m}$ (but is not a Joule)

Several useful conversion factors are as follows:

- 1 inch = 2.54 cm (exact digits)
- 1 gram = 980.7 dynes = 980.7 g-cm/s²
- 1 Newton = 100,000 dynes
- 1 psi = 0.07031 kg/cm^2
= 6.89428 kPa
- 1 lb/ft³ = 0.157093 kN/m^3
- 1 ft³ H₂O = 62.4 lb/ft³ (pcf)
- 1 m³ H₂O = 9.807 kN/m^3
- 1 g/cm³ = 9.807 kN/m^3
- 1 gal (US) = 3.785 liters = 0.003785 m³

Fundamental Definitions

Referring to Fig. I-1a, we have a volume of soil removed from a field location. Further, it is assumed that the soil was removed in the form of a cube with lateral dimensions of 1 cm (volume = 1 cm³). Actually it would be difficult to do this in practice, but for illustrating the volume-weight relationships which follow, we shall assume that we could remove this soil volume as a perfect cube. This cube is made up of a soil skeleton with water and air in the interstices, or *pores*, between contact points of the soil particles (Fig. I-1b). It should be evident that, depending on the location of the cube of soil in the field (in situ) and on climatic factors, the quantity of water and air can vary from all the pores full of water and no air present to all the pores full of air and no water. Depending also on the instantaneous temperature, the water could be present as ice or as an ice-water mixture.

For the purpose of better visualizing the make-up of the cube of soil, let us first drain the cube of all the water present and place it in a container. Next we heat the soil skeleton until it melts and flows together to form a solid, nonskeletal (nonporous) mass to occupy the volume V_s of a container with a total volume of 1 cm³, as in Fig. I-1c. Notice that the soil skeleton occupied a volume of 1 cm³. The actual volume of soil solids, V_s , is less than 1 cm³. Now we pour the previously collected volume of water V_w into the container on top of the solids volume (Fig. I-1d). If the pores of the soil skeleton had been full of water, it should be evident that the volume of soil solids plus the volume of pore water would fill the 1 cm³ container. Since in this illustrative case they were not full of water, the remainder of volume required to fill the cube must be the air volume V_a present in the original soil skeleton.

The assumptions listed below will be made for convenience in the following material:

1. Soil has weight.
2. Air has no weight (for the volumes we shall be using).

3. Water has weight. Generally we shall take this as 1 g/cm^3 (9.807 kN/m^3), although this value is correct only at a temperature of 4°C . At temperatures of 18 to 25°C , the weight ranges from 0.9986 to 0.9971 g/cm^3 (see Table 6-1).

The following symbols will be used in the definitions to follow:

- e = void ratio
- G = specific gravity of any substance
- G_s = specific gravity of the soil solids
- G_w = specific gravity of water
- n = porosity
- w = water content
- S = degree of saturation
- W_s = weight of dry soil solids
- W_t = total weight of a soil mass including any water present
- W_w = weight of water present in a soil mass
- V_a = volume of air present in a soil mass
- V_s = volume of soil solids in a soil mass
- V_t = total volume of a soil mass = $V_s + V_w + V_a$
- V_w = volume of water in a soil mass
- V_v = volume of voids in a soil mass = $V_w + V_a$
- γ = unit weight of a material (g/cm^3 or kN/m^3)
- γ_d = dry unit weight of soil
- γ_s = saturated unit weight of soil
- γ_w = unit weight of water

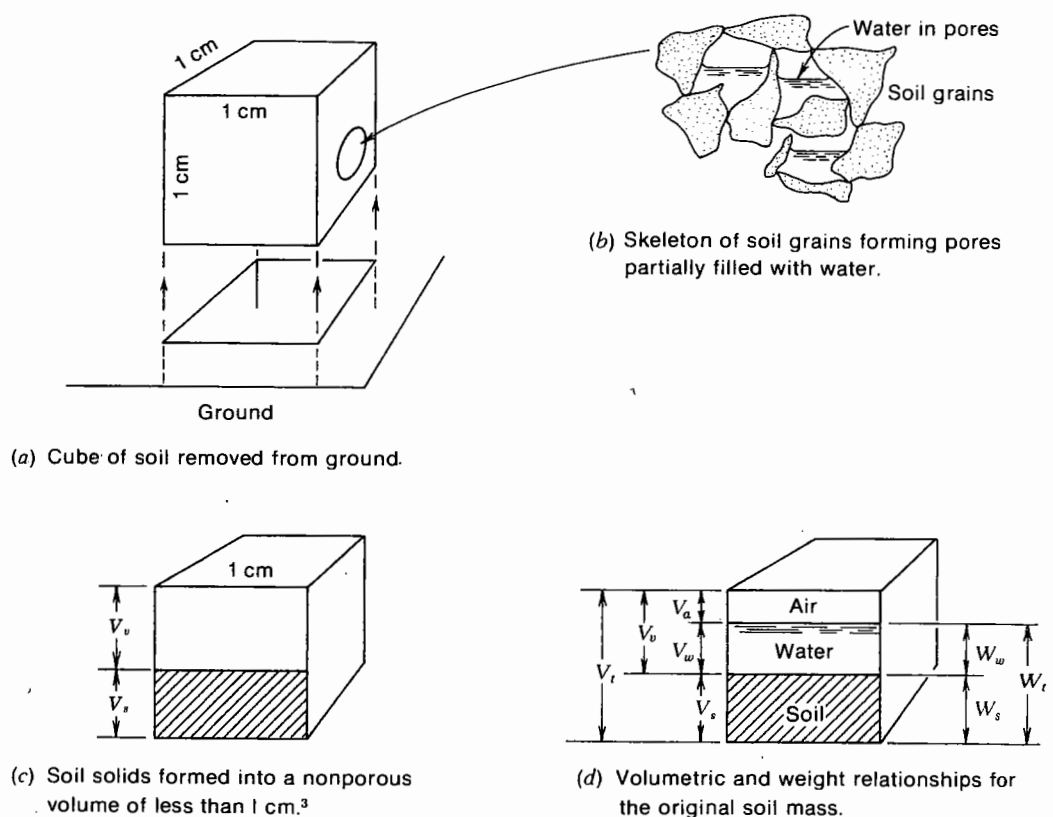


Figure I-1
Volumetric-gravimetric relationships.

Referring to Fig. I-1d, we define the *void ratio* e as

$$e = \frac{V_v}{V_s} \quad (\text{I-1})$$

The void ratio is usually expressed as a decimal.

The *porosity* n is defined as

$$n = \frac{V_v}{V_t} \quad (\text{I-2})$$

The porosity may be expressed either as a decimal or as a percentage; decimal usage is preferred.

By substituting Eq. (I-1) into Eq. (I-2), we can obtain the void ratio in terms of the porosity as

$$e = \frac{n}{1 - n} \quad (\text{I-3})$$

The *degree of saturation* S is defined as

$$S = \frac{V_w}{V_v} 100 \quad \text{percent} \quad (\text{I-4})$$

Notice that the range of S is from 0 to 100 percent and if

$S = 0\%$, the soil is *dry*

$S = 100\%$, the soil is *saturated*

The *unit weight* γ of a soil is

$$\gamma = \frac{W_t}{V_t} \quad (\text{I-5})$$

This definition is also applicable to other substances such as iron, copper, coal, and oil. If the soil is *dry* ($S = 0\%$), note that $W_t = W_s$ and we compute the *dry unit weight* γ_d as

$$\gamma_d = \frac{W_s}{V_t}$$

If $S = 100$ percent, $W_t = W_s + W_w$ and using Eq. (I-5) we compute the *saturated unit weight* γ_s .

If $0 < S < 100$ percent, W_t is still computed as $W_t = W_s + W_w$ and Eq. (I-5) gives the *wet unit weight* of the soil. Generally one must look at the context of a given problem to see which unit weight is being described or is to be determined, since, as the preceding material indicates, more than one unit weight value may be computed for a given soil depending on the value of weight used in the numerator of Eq. (I-5).

From fundamental considerations, the volume of any material is

$$V = \frac{W}{G \gamma_w} \quad (\text{I-6})$$

Thus, referring to Fig. I-1d, the volume of soil solids V_s is

$$(I-1) \quad V_s = \frac{W_s}{G_s \gamma_w} \quad (I-6a)$$

The volume of water V_w is

$$(I-2) \quad V_w = \frac{W_w}{G_w \gamma_w} = \frac{W_w}{\gamma_w} \quad (I-6b)$$

since G_w and γ_w are both approximately 1.0 for water at usual laboratory and field temperatures.

Water content w is defined in Experiment No. 1. For convenience, the equation definition of water content is repeated here:

$$w = \frac{W_w}{W_s} \quad \text{percent}$$

The method of determining the specific gravity G_s of a soil is given in Experiment No. 7.

With appropriate assumptions and substitution of equations into each other, various relationships can be derived. For example:

Given quantities	To find	Derived expression
γ_d, G_s	e	$e = \frac{G_s}{\gamma_d} - 1$
$G_s, w (S = 100\%)$	n, e	$n = \frac{w G_s}{1 + w G_s}$ $e = w G_s$
G_s, e	γ_d, γ_s, n	$\gamma_d = \frac{G_s \gamma_w}{1 + e}$ $\gamma_s = \frac{G_s + e}{1 + e} \gamma_w$ $n = \frac{e}{1 + e}$

Generally: $Se = wG_s$

Other relationships can be derived, but they will be left as an exercise for the student.

Example

Given the following:

Weight of container + wet sand = 248.5 g
 Weight of container + dry sand = 231.2
 Weight of container = 63.7
 Volume of container = 100 cm³

Find: (1) w , percent; (2) dry unit weight; (3) wet unit weight; and (4) void ratio if $G_s = 2.68$.

SOLUTION

$$1. \quad w = \frac{W_w}{W_s} \times 100 = \frac{248.5 - 231.2}{231.2 - 63.7} \times 100 = \frac{17.3}{167.5} \times 100 = 10.3\%$$

$$2. \gamma_d = \frac{W_s}{V_t} = \frac{167.5}{100} = 1.675 \text{ g/cm}^3 = 1.675 \times 9.807 = 16.43 \text{ kN/m}^3$$

$$3. \gamma_{wet} = \frac{W_{wet}}{V_t} = \frac{184.8}{100} \times 9.807 = 18.12 \text{ kN/m}^3$$

Also $W_t = W_s + W_w = W_s + wW_s$ (from step 1). Now divide by V :

$$\frac{W_t}{V_t} = \frac{W_s}{V_t} + \frac{wW_s}{V_t} = \gamma_{wet} = \gamma_d + w\gamma_d$$

$$\gamma_{wet} = 16.43 + 0.103(16.43) = 18.12 \text{ kN/m}^3 \quad \text{as before}$$

$$4. e = \frac{V_v}{V_s} \quad \text{but} \quad V_s = \frac{W_s}{G_s \gamma_w} = \frac{167.5}{2.68(1)} = 62.5 \text{ cm}^3$$

$$V_v = V_t - V_s = 100 - 62.5 = 37.5 \text{ cm}^3$$

$$e = \frac{37.5}{62.5} = 0.60$$

Laboratory Procedure

Laboratory equipment is expensive; this can be more properly appreciated by looking at a current copy of a supplier's price list. The costs of various items of equipment may be indicated at various times throughout the term. Equipment may be damaged by careless handling, and damaged equipment may yield serious test errors. Scales are especially susceptible to damage. For this reason, they are not to be moved to other laboratory locations except under the instructor's supervision. The triple-beam balances used primarily for water-content determinations should be locked at the end of each laboratory period. The balances should be checked for zero reading before using, and any series of weighings should be taken on the same balance to avoid zero errors between two balances. Small quantities are used in certain tests, and in such tests weighing errors can be very important.

The drying ovens are preset to a temperature of 105 to 110°C, and the thermostats should not be manipulated without instructor authorization as it takes considerable time to stabilize the oven temperature so that the thermostats can be set properly. It is essential in soil mechanics work that this particular value of oven temperature be maintained. Oven-dried samples will be removed from the ovens by the appropriate persons no later than 24 h after placing the samples in the oven. The instructor reserves the right to remove and discard any samples left in the ovens longer than this. This rule allows the ovens to be turned off when they are not needed.

Each group will clean up and be responsible for its work area and equipment. Remember that others also use the equipment and if the weight of a piece of equipment is needed and the item has to be washed and dried, valuable laboratory time will surely be lost.

Laboratory Reports

A laboratory report is required for all projects unless otherwise stated by the instructor. This report will be in a folder (one report to a folder) on which the following information is shown:

1. Name of student
2. Title of project
3. Course number and laboratory section
4. Laboratory group number
5. Date of project and date of report submission

The report will be typewritten or *neatly hand-lettered* with a ball-point pen and will consist of the following material in the given sequence:

1. *Flyleaf* showing title of project, name of student, course number, date of work, date of submission of report, and the names of the other group members.
2. *Introduction*—a brief (one-half page or less) summary of the objectives of the work.
3. *Discussion of the work*—including any special techniques used or changes in laboratory manual instructions. Any equipment limitations or possible sources of error should be discussed. If your results are not very good, give *your* analysis of the probable cause. If you use any equations other than basic definitions, include the equations here and note that their derivations are shown in the sample calculations. The “discussion” should generally take less than $1\frac{1}{2}$ pages.
4. *Conclusions*—a brief summary and tabulation of the project results. If any improvements can (or you think could) be made, these should be listed under this heading. Do not say you got a lot (or nothing) out of the experiment; this is not considered a “conclusion.” This part of the report should be about one-half to one page long, depending on the amount of information to be presented. Be sure to tabulate your test results in this section of your report.
5. Show any graphs next and follow these with your collected data, which will generally be on one of the data sheets furnished. The data sheet is to be completely removed from the data sheet section of this manual and detached from the serrated edge along the perforated line. *Do not recopy your data*—get in the habit of taking neat (or at least reasonably neat) laboratory notes. The report is to contain original data; if it is recopied, list the group members who took the original data.
6. *Sample calculations*—it is not necessary to show every calculation, but one calculation which is typical should be shown. If the data are to be used in several computations, use the same data for all the computations shown. Be sure to show any derivations required to obtain equations for use in the data reduction.

The report is to follow good technical report writing form, including the citing of any references used. Do not use the first person (I, me, we, our, etc.) in writing a technical report. Do not write such statements as “I found that . . .” or “My group found . . .”; instead use “It was found that . . .” Use correct spelling—when in doubt, consult the dictionary. Try to use good sentence construction, and do not change from past tense to present tense in the same sentence or paragraph. Many of these problems can be avoided by:

1. Writing the report and then reading it over a day or so later. A neatly marked-through word or two will be better received by the instructor than a report which sounds ridiculous. It will also indicate that you went over the report prior to submitting it.
2. Not writing in such a hurry that you tend to drop letters off the ends of words or spell a word as it sounds.
3. Thinking about what you are going to write and being brief. A few well-put-together sentences are far more impressive than a long, poorly written report which says nothing.
4. Obtaining (perhaps from the library) a text on technical report writing and consulting it for writing tips.

A primary purpose of the report is to give the instructor an indication of what you learned from the project. Other major benefits are obtained from practice in report writing and presenting engineering data. While you are graded primarily

on the project and the presentation of the data (and not report writing), a poorly written report will generally result in a lower grade than a well-written report.

Although students may not fully appreciate at this time the formality involved in the reports, they should realize that report writing is an essential part of the work of most engineers. Further, if they do not use soil mechanics immediately upon graduation and several years later are required to do work in this area, they will have some refresher material ready at hand—old laboratory reports—which, if well written, will be easy to follow.

Drawing Graphs

A curve should be legible, neat, and easily understood. Graph sheets, both regular grid and semilogarithmic of the necessary number of cycles, are furnished in the data sheet section. If you use other graph paper, it *must be* 21.5×28 cm in size (currently $8\frac{1}{2}$ by 11 in in the United States). It is usually undesirable to paste several pieces of semilog paper together to obtain the required number of cycles because of the resulting page folds when the report is bound. The use of graph paper in centimeter divisions is encouraged, but unfortunately, as of this writing (1977), standard paper in the United States is $8\frac{1}{2} \times 11$ in.

When drawing the curves, always place them on the graph sheet so that the left and the lower margins are both at least 2 cm wide. All lettering on the axes will be on the ruled portion of the sheet, and the line ruling should be used as lettering guidelines. Always use as large a scale as possible, but one which is easy to plot and/or read. For example, scales of either 5 (or 10) divisions/cm plot well in multiples of 1 and 5 units (sometimes 4), but scales of 3, 6, or 7 units/division are awkward for both plotting and reading.

Show a title block (Fig. I-2) on all graphs which includes:

1. Title of project (*Example: Dry density vs. water content*)
2. Type of soil (*Example: Brown silty clay $w_L = 42.1\%$; $w_p = 21.3\%$*)
3. Date of work (*Example: 10 January 1979*)
4. Scale (if appropriate)
5. Name of person preparing the graph

The title block will always be located in the upper right-hand or lower right-hand corner of the graph. When the results of several tests are shown as curves on a single graph, a legend should be used to identify the data plotted from the different tests, as shown in Fig. I-2. Several curves on a sheet generally make a better presentation if shown using lines of different colors or types (solid, dashed, dash-dots, etc.).

Show plotted points not as dots but as illustrated in Fig. I-2 (note that the curve lines do not go through the plotted points) so that anyone can easily check your interpretation of the plotted data. Always draw smooth curves using a French curve (never freehand) unless the instructor tells you to use straight lines. All graphs will be oriented to read with the binding to the left side or top of the displayed page.

Before plotting graphs, give thought to what information the graph is to present. Is it qualitative (relative) or quantitative (numerical) or both? Obviously, quantitative information requires a better scale than qualitative information. How have others (in textbooks, etc.) presented the same type of data; i.e., does the ordinate or abscissa best control the plot? As an example, plotting a curve which is asymptotic generally (but not always) displays better with the asymptote horizontal rather than vertical.

The values to be obtained from the plot of experimental data should always be shown on the graph. If these values are used to compute a constant (for

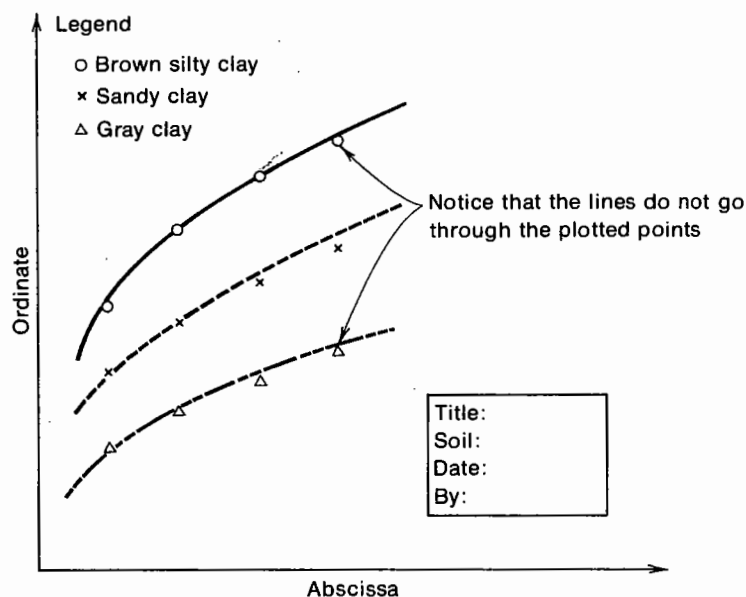


Figure I-2

Method of displaying test data on a graph when more than one test is to be shown. Note the use of both symbols and line types to separate the information. Always include some kind of title information for identification purposes.

example, the modulus of elasticity), show both the computation and the constant on the graph.

Laboratory Practice

The procedures outlined in the following tests are reasonably standard. For example, distilled water is often specified for use in the ASTM¹ versions of the tests. This can be omitted and ordinary tap water used for the tests (at the discretion of the instructor), yielding, in general, a negligible error. Methods of soil preparation and length of testing may be modified from the ASTM Standards in the interest of saving student time. A copy of the current ASTM Standards Part 19 (ASTM publishes standards for materials testing and specifications in 48 parts; all the soil tests which have been standardized to date are found in Part 19) will be available in the soils laboratory or library for the interested student. The soil tests outlined in the following pages of this manual list the correct ASTM and AASHTO² Standards designation (if there is one) for reference convenience. For example, the ASTM Standards designation for the test in Experiment No. 1 is D2216-71, where D2216 identifies the test and 71 is the year of adoption of the test standardization. If a letter T follows the year of adoption, this indicates that the test has been tentatively standardized.

Soil is highly variable in nature, and this variability cannot be controlled to any great extent by engineers, who must work with the material as they find it, although it may be possible to alter its structure or change its composition by mixing it with imported materials. Soil tests are for the purpose of identifying (or classifying) the material, determining certain of the physical properties of the material, and establishing control criteria for the material.

¹American Society for Testing and Materials, 1916 Race Street, Philadelphia, Pa. 19103.

²American Association of State Highway and Transportation Officials, 341 National Press Building, Washington, D.C. 20004.

Since it is obviously impossible to test the entire soil mass and since soil is a variable quantity, it is necessary to perform a few tests on small quantities of soil and extrapolate the results to the entire soil mass. For the tests to be valid for the soil mass, they must be on samples which are *representative* of the soil mass. Obtaining truly *representative* samples requires great care in sampling operations. Bags of samples from the field are collected by the field personnel to be representative. The smaller samples for individual tests should be representative of the bag of soil; thus picking out a few lumps from the top of the bag is very probably not going to yield a representative sample. Keep constantly in mind the fact that you are or will be, as a soils engineer, estimating with tests on a few grams of material the soil performance of huge masses of soil.

The electronic pocket calculator has made the slide rule obsolete for engineering computations and tends to introduce a fictitious precision to computed results. The user of this text should be aware of the data limitations when making computations. The only computations in this manual which justify using two decimal places (to the nearest 0.01) in the final answer are the specific-gravity test (Experiment No. 7) and density tests (Experiments 9 and 10). All other work can hardly justify more than one decimal (to the nearest 0.1). Therefore, even when averaging two or more test results, round off and report the answer to the nearest 0.1 (or to 0.01 for the 3 tests cited).

Engineering Units and Data Collection

When using the data sheets (from the data sheet section) for your laboratory data assembly *always insert the units* of the dial gages, load rings, whether a load cell is used, or any other information which may be needed later when reducing the data to complete the test. *Never omit these units* as you may later forget or, more importantly, if you need to refer to your report several weeks or months later, you will immediately see the units used and what you did to obtain the test results shown. This will be especially critical in the next few years as laboratories renew wornout equipment with SI/metric equipment so that both types of equipment may be used when you are taking data.

Experiment No. 1

WATER-CONTENT DETERMINATION

References

ASTM D2216-71 (ASTM Standards Part 19)

Equipment

Moisture cans (tin or aluminum)
Oven with accurate temperature control

General Discussion

Water-content determination is a routine laboratory test to determine the amount of water present in a quantity of soil *in terms of its dry weight*. As a definition,

$$w = \frac{W_w}{W_s} 100 \quad \text{percent} \quad (1-1)$$

where W_w is the weight of water present in the soil mass and W_s is the weight of soil solids. One could define the water content as the ratio of the weight of water present to the total weight (i.e., weight of water plus soil); however, this would give a quantity in the denominator of the fraction which would depend on the amount of water present:

$$w' = \frac{W_w}{W_s + W_w} = \frac{W_w}{W_t} \quad (1-2)$$

This is not desirable since the water content would then be related to a varying quantity rather than a constant quantity. This is easily seen, as W_w appears in both the numerator and the denominator of the fraction: Eq. (1-2). Water content is sometimes expressed on a volume basis as

$$\Theta = \frac{V_w}{V_s + V_v} = \frac{V_w}{V_t} \quad (1-3)$$

which, through suitable manipulation, can also be written as

$$\Theta = w \gamma_d \quad (1-4)$$

where V_w = volume of water present in the soil mass

V_v = volume of soil voids

V_s = volume of soil solids

w = water content, from Eq. (1-1)

γ_d = dry density of soil (bulk)

This definition of water content is rarely used in the United States.

Procedure

1. Weigh a tin or aluminum cup, including the cover. Be sure to identify the container and its lid. The moisture cups are available in several sizes, with the 5-cm diameter by 3-cm height and the 6.4-cm diameter by 4.4-cm height being rather popular.
2. Place a representative sample of wet soil in the cup and determine the weight of wet soil + cup. If the weight is determined immediately, it is usually not necessary to place the lid on the cup. If there is a delay of 3 to 5 min or more, put the lid on the cup to retain the moisture and place the cup under a damp paper towel to maintain the humidity in the vicinity of the cup.
3. After weighing the wet sample + cup, *remove the lid*—it is usual practice to place it on the bottom of the cup—and place the sample in the oven.
4. When the sample has dried to a constant weight, obtain the weight of the cup + dry soil. Be sure to use the same scales for all weighings.
5. Compute the water content w . The difference between the weight of wet soil + cup and the weight of dry soil + cup is the weight of water W_w which was present in the sample. The difference between the weight of dry soil + cup and the weight of cup is the weight of soil W_s and

$$w = \frac{W_w}{W_s} \times 100 \quad \text{percent} \quad (1-1)$$

The soil is to be dried in the oven at a drying temperature of $110 \pm 5^\circ\text{C}$ to a *constant weight*; i.e., so long as water is present to evaporate, the weight will continue to become smaller with successive weighings. It is usually not practical to make several weighings to determine if a constant weight state has been obtained; what is usually done is to assume that after an oven-drying period of 12 to 18 h (often overnight), the sample is at a constant weight state and that weight is used for the weight of dry soil + cup. Experience indicates that this method of sample drying is quite adequate for the small samples in routine laboratory work.

It is usual to remove the water-content samples from the oven and to weigh them immediately (use a pair of tongs or asbestos glove as they are quite hot). If for some reason it is not feasible to weigh the dry samples immediately, the cup lid should be put on as soon as the cup is cool enough to handle and/or the can of dry soil should be placed in a desiccator so that the soil does not adsorb water from the laboratory atmosphere.

To provide reasonably reliable water-content determination, the following minimum wet-soil sample weights (*representative samples*) are recommended:

Maximum size of soil particles (95–100% passes the given sieve)	Recommended minimum sample weight (g)
No. 4 (4.75 mm)	100
No. 40 (0.420 mm)	10 to 50
12.5 mm	300
50.0 mm	1,000

The oven temperature of 110°C is too hot for certain organic (peaty) soils, for soils containing appreciable amounts of gypsum or other minerals, certain clays, and some tropical soils. These soils contain loosely bound water of hydration, or molecular water, which can be lost at this high a temperature, resulting in a change in the soil characteristics—notable the Atterberg limits of Experiment No. 3 as well as grain size and specific gravity. ASTM suggests an oven temperature of 60°C for drying these soils.

Experiment No. 2

FIELD COLLECTION OF A SOIL SAMPLE

References

AASHTO T86-70

ASTM D420-69

Read "Water-Content Determination" (Experiment No. 1)

Objectives

This project is for the purpose of:

1. Introducing the student to one method of obtaining disturbed soil samples
2. Enabling the student to obtain an indication of the variation of natural soil moisture with depth
3. Collecting information to plot a simple soil profile
4. Obtaining a bag of soil for future laboratory testing¹

Equipment

5-cm or 7.6-cm hand auger (or small gasoline-powered continuous-flight auger)
Extension rods for the auger to obtain borings of at least 6 m
2 sample bags per party
12 moisture cans per party (obtain empty weights before going to field)
Wrenches to add on auger extension rods
30-m tape to locate borings

Procedure *This Will Be a Group Project*

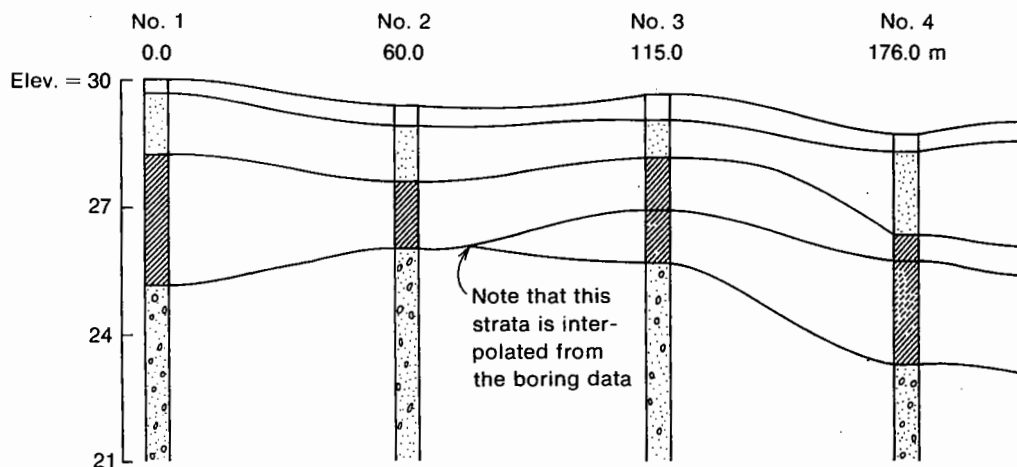
1. Each group is to auger a hole at least 6 m deep.
2. Take two moisture-content samples at each 1 m of boring depth and where visual soil-stratum changes occur. Place the lids on the moisture cans immediately on obtaining the samples so that the moisture loss is minimized. Upon return to the laboratory, weigh the cans of moist soil, *remove the lids*, and place the samples in the oven for drying. Return to the laboratory the following day and weigh the dried soil and compute the water content. Average the two values of water content as the reported water content at each elevation.
3. Collect two bags of clayey soil per group (or enough material to yield at least 10 kg of air-dry soil). This will be taken back to the laboratory, labeled, and stored for later soil testing, except that 5 kg will be immediately placed in a large pan for air drying for use in the laboratory period for the next week.
4. For this laboratory report each group will, on the day following the field work, place on the blackboard in the soil laboratory the profile and location of their boring. Each student will use these data to plot a simple soil profile using a scale of 2 cm = 1 m vertically and 2 cm = 15 m horizontally on a sheet of 21.5 × 28 cm graph paper. Use a visual description of the soil in each stratum (e.g., gray, sandy clay; black topsoil, gravelly sand). Each student will also

¹If this experiment is not done due to time or weather, it is suggested that the instructor obtain a quantity of soil from a construction site, during good weather and prior to classes, and store it for future laboratory testing.

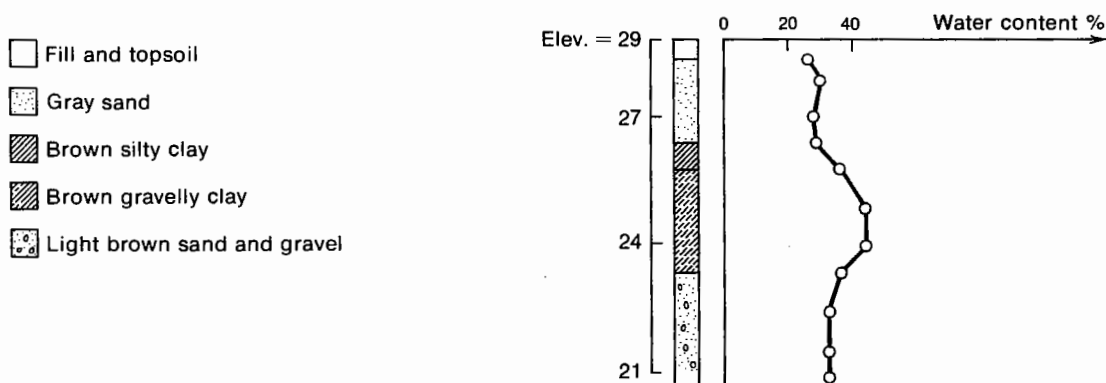
present on a second sheet of 21.5×28 cm graph paper a profile of the individual boring with a plot of water content with depth as shown in Fig. 2-1. A legend similar to that shown in the figure should be used.

Use water content data and graph sheets from those furnished in the back of this manual.

In the brief report to be submitted for this project, can you draw any conclusions on the variation of water content with depth—especially if any recent rainfall has occurred?



(a) Soil Profile from boring information.



(b) Water content with depth for boring No. 4.

Figure 2-1

Typical boring-log presentation. Note the plotting of the ground-surface profile. Some persons (Experiment No. 2) plot using the ground surface as reference for each boring. Exact details may vary somewhat from laboratory to laboratory. All dimensions shown are in meters.

Experiment No. 3

LIQUID AND PLASTIC LIMITS OF A SOIL

References

- AASHTO T89-68 and T90-70
ASTM 423-66 (Liquid Limit) and D424-59 (Plastic Limit)
ASTM (1960), Papers on Soils: Symposium on Atterberg Limits, *Special Technical Publication (STP) no. 254*, pp. 159-226 (with numerous references).
Casagrande, A. (1932), Research on the Atterberg Limits of Soils, *Public Roads*, vol. 13, no. 8, October, pp. 121-136.
Fang, H. Y. (1960), Rapid Determination of Liquid Limit of Soils by Flow Index Method, *Highway Research Board Bulletin no. 254*, pp. 30-35 (also contains earlier references).
Nuyens, J. G. E., and R. F. Kockaerts (1967), Reliable Technique for Determining the Plastic Limit, *Mater. Res. Stand., ASTM*, vol. 7, no. 7, July, pp. 295-299 (see also December 1968, pp. 17-19).
Seed, H. B., R. J. Woodward, Jr., and R. Lundgren (1964), Fundamental Aspects of the Atterberg Limits, *J. Soil Mech. Found. Div., ASCE, SM6*, November, pp. 75-105 (see also *SM 4*, July, pp. 107-131, same authors).

Objective

To introduce the student to the procedure for determining the liquid and plastic limits of a soil.

Equipment¹

- Liquid-limit device with grooving tool (Fig. 3-1)
- Moisture cups
- Plastic-limit plate (optional)
- Soil-mixing equipment (porcelain dish, spatula, plastic squeeze bottle to add controlled amount of water)
- Balance sensitive to 0.01 g
- Sieve, pan, and lid (US No. 40, BS No. 36, AFNOR No. 27, or DIN No. 400; see Table 5-1)

Figure 3-1

Usual liquid- and plastic-limit-test equipment. Shown are the following: No. 40 sieve and pan; liquid-limit device with both ASTM (on right) and Casagrande grooving tools; plastic squeeze bottle for dispensing controlled quantities of water; porcelain dish and spatula for carefully mixing soil; plastic-limit-test glass plate and an optional 3-mm welding rod for visual comparison of plastic-limit thread.



¹Use a stock laboratory soil run through a 10-mm (3/8") wire screen and stored in a 20-gal garbage can. Soil should be obtained prior to beginning of course; a large enough supply should be available for a laboratory section to perform Experiments No. 3, 6, 7, part of 8, 9, and 14. A small portion can be air-dried for several days prior to Experiment No. 3, with enough left over for Experiments No. 6 and No. 7.

General Discussion

The liquid and plastic limits are two of five "limits" proposed by A. Atterberg, a Swedish agricultural scientist (ca. 1911). These limits are:

1. *Cohesion limit*—that moisture content at which soil crumbs just stick together.
2. *Sticky limit*—that moisture content at which soil just sticks to a metal surface such as a spatula blade. This would have some significance to the agricultural engineer since it is related to soil sticking to the moldboard of a plow or disc in cultivating soil.
3. *Shrinkage limit*—that moisture content below which no further soil volume reduction (or shrinkage) occurs. The method of determining this moisture content is presented in Experiment No. 4.
4. *Plastic limit*—moisture content below which the soil is nonplastic.
5. *Liquid limit*—moisture content below which the soil behaves as a plastic material. At this moisture content, the soil is on the verge of becoming a viscous fluid.

The liquid and plastic limits have been widely used all over the world, primarily for soil identification and classification. The shrinkage limit is useful in certain geographical areas where soils undergo large volume changes when going through wet and dry cycles. The potential-volume-change problem can often be detected from the liquid- and plastic-limit tests. The liquid limit is sometimes used to estimate settlement in consolidation problems (Experiment No. 13), and both limits may be useful in predicting maximum density in compaction studies (Experiment No. 9). The two methods of soil classification presented in Experiment No. 8 incorporate the use of the liquid and plastic limits.

The cohesion and sticky limits are used very little worldwide. In fact, only recently has it become widely known that Atterberg proposed five and not three Atterberg limits.

The relative locations of the shrinkage limit w_s , plastic limit w_p , and liquid limit w_L are shown on the water-content scale in Fig. 3-2.

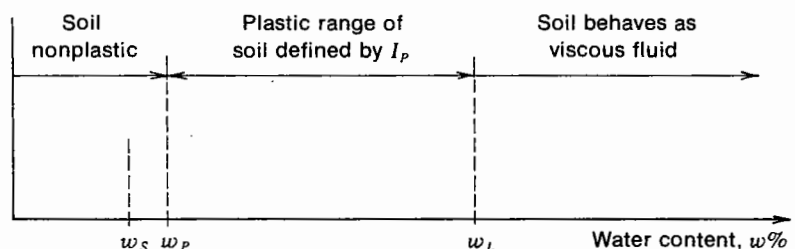


Figure 3-2

Relative locations of shrinkage, plastic, and liquid limits on a water-content scale. Note that the w_s location may vary for some soils to the right of w_p .

In order to place definite, reproducible values on these limits, it was proposed that the *liquid limit be arbitrarily defined* as that water content at which a pat of soil placed in a brass cup, cut with a standard groove, and then dropped from a height of 1 cm will undergo a groove closure of 12.7 mm when dropped 25 times. Several variables affect the liquid-limit test or the number of blows required to close the standard groove 12.7 mm, including

1. Size of soil pat in brass cup (thickness and quantity)
2. Rate of blows (should be 120 rpm)

3. Time soil is in cup prior to beginning test blow count and how clean cup is prior to adding soil for the test
4. Laboratory humidity and speed of performing test
5. Type of material used for liquid-limit device base (commonly micarta or hard rubber)
6. Accuracy of height-of-fall calibration (should be exactly 1 cm)
7. Type of grooving tool (either ASTM or Casagrande type)
8. Condition of liquid-limit device (worn pins, loose connections, etc.)

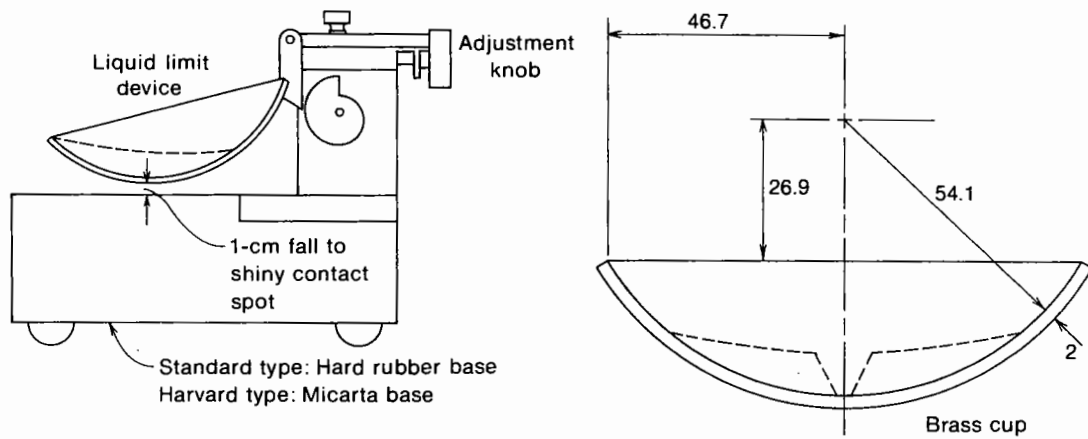
These variables are all operator-controlled. The liquid limit (w_L) is also markedly affected by the type of soil and other factors to be considered. To attempt to reduce the test variables, standard devices are used together with standard grooving tools. One of the grooving tools (Fig. 3-3b) is the ASTM standard tool; the other tool shown was developed by Casagrande (1932) and has the advantage of tending to control the depth of the soil pat in the cup. The ASTM grooving tool is better for soils which have low liquid limits and are difficult to groove, such as sandy and silty materials. For these soils, it may even be necessary to partially form the groove using a spatula, after which the groove can be properly shaped using one of the standard grooving tools.

To control the rate of blow occurrence, the crank controlling the blows is rotated at 120 rpm, that is at a rate of 120 blows per minute.

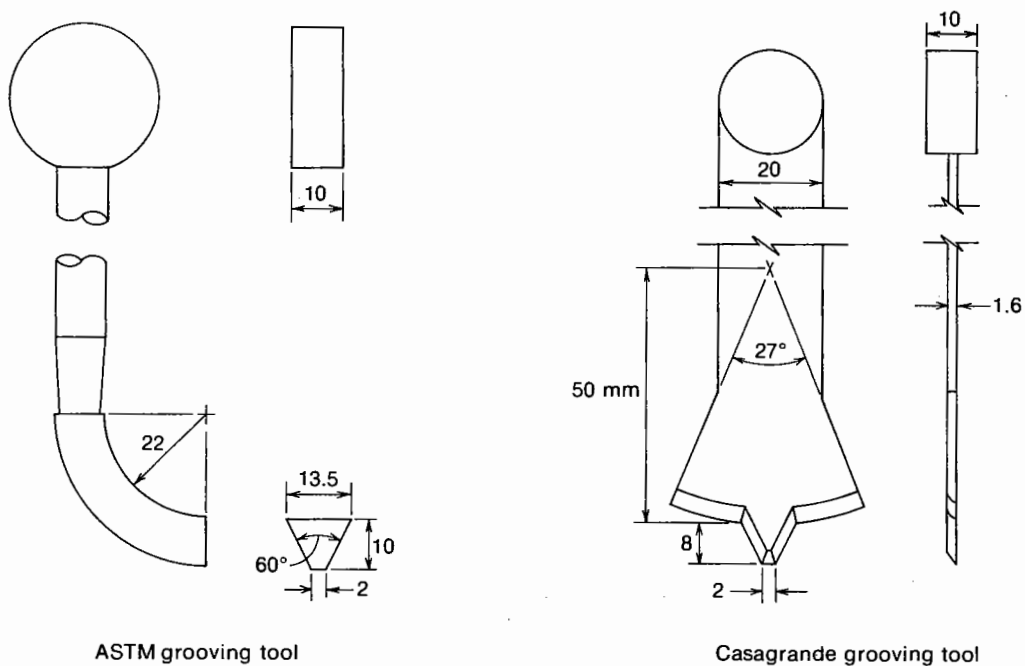
The ASTM Standard for this test stipulates the use of distilled water when mixing the soil for testing. However, for most laboratory work ordinary tap water is satisfactory.

The Atterberg limits tests are performed on soil passing No. 40 sieve. Ordinarily, soil obtained from the field will be too wet to pass through the sieve openings of the No. 40 sieve. It is permissible (and allowed by ASTM D421-58, "Dry Preparation of Soil Samples for Grain-size Analysis and Determination of Soil Constants") to *air-dry* the soil to obtain the (—) No. 40 (0.425 mm) soil fraction in most cases. Oven drying of the soil to obtain a material which can be sieved through the No. 40 sieve will generally lower the values of both the liquid and plastic limits and should never be done. Research by the author as well as that reported by Casagrande (1932) has shown that ordinary air drying of many soils will lower the liquid limit by as much as 2 to 6 percent. To avoid this problem, it is sometimes possible to use the material, at the field moisture content, which appears to pass the No. 40 sieve based on a visual inspection. Research also indicates that most air-dried soils will regain their original limits if the soil is mixed with water and allowed to cure for 24 to 48 h.

It was stated previously that the liquid limit is defined for the water content at which 25 blows close the standard groove 12.7 mm. It would be fortuitous indeed if one could find this water content within a reasonable length of time. Since this is next to impossible, some other means must be used to find this particular value of water content. It has been found that if one makes a graph by plotting blow counts to a logarithmic scale vs. water content to a natural scale, the locus of points for any one soil tends to fall on a straight line. With this phenomenon established, it becomes relatively easy to find the liquid limit indirectly. All one has to do is obtain three to six points at different water contents together with the corresponding blow counts (obviously, distributed on both sides of the blow count of 25 for best results), plot these data to a semilogarithmic scale, and from the blow count of 25 read from the resulting curve projection the water content (liquid limit) for the soil. It appears that the linear relationship only holds across one cycle of the semilog plot (10 to 100 blows). It should be self-evident that the closer the plotted points are to the blow count of 25, the more reliable the value of liquid limit obtained from the curve since some extrapolation is inevitably involved.



(a) Construction details and dimensions of the Liquid limit device.



(b) Grooving tools

Figure 3-3
Liquid-limit test equipment. All dimensions are in millimeters.

A term to indicate:

1. The percentage of clay in the (–) 40 fraction of soil as used for the Atterberg limits, and
2. The potential swell and shrinkage (volume change) of a soil, with larger values indicating increasing potential

is the *activity* of a soil. *Activity* of a soil is defined as

$$A = \frac{\text{plasticity index}}{\text{percentage of clay in mixture} - C_o} \quad (3-1)$$

The constant C_o is 0 or 9, depending on whether Skempton's work or the work of Seed et al., (1964) is used. The plasticity index I_p is defined in Fig. 3-1. The percentage of clay in the mixture [(-) No. 40 fraction] used in the above equation is based on the percentage of soil grains 0.002 mm and smaller in size, which is not universally agreed upon as the upper limit of clay sizes.

The smaller the value of A in Eq. (3-1), the smaller the potential volume change of the soil. The approximate range of A is from 0.3 to about 5.5 and depends on the type of clay minerals present; e.g., a predominantly kaolinite clay has a lower activity than illite or montmorillonite clays.

The liquid limit is a measure of the shear strength of a soil at some water content. The liquid limit is analogous to a shear test, and Casagrande (1932) has found that each blow to close the standard groove corresponds to about 1 g/cm² of shear strength. Others have obtained similar results so that one might say that the liquid limit represents for all soils a constant shear-strength value of between 20 and 25 g/cm². That the liquid limit increases as the grain sizes of the soil mass decrease is another fundamental observation from research.

The plastic limit, besides being the lower boundary range of plastic behavior of a soil, tends to increase in numerical value for decreasing grain sizes. For equal grain sizes, the plastic limit tends to increase for that soil with the more scalelike particles. The plastic limit is also a measure of the shear strength of a soil.

The research by Seed et al. (1964) has also shown that the A line of the Casagrande classification diagram shown in Fig. 8-2 is essentially correct. Casagrande proposed this diagram based on plotting large numbers of values from many clays in the early 1940s. Now that it has been established that both the w_L and w_P are dependent upon the percentage of clay in the (-) No. 40 fraction of the soil, the following linear equation can be written for the liquid limit:

$$w_L = K(PC - a) \quad (3-2)$$

Rewriting Eq. (3-1), we obtain $I_p = A(PC - C_o)$

where PC = percentage of clay sizes (smaller than 0.002 mm) in the soil fraction passing the No. 40 sieve.

K, a = constants to be determined

Eliminating PC in the above equations and substituting new constants, we obtain

$$I_p = N(w_L - b) \quad (3-3)$$

Casagrande's classification diagram uses $N = 0.73$ and $b = 20$.

Since the semilogarithmic plot of water content vs. blow count (\log) is a straight line, the equation of this line can be written in the general form

$$w = -F_i \log N \pm C \quad (3-4)$$

where w = moisture content at N blows

F_i = flow index, the change in moisture content Δw across one cycle of the plot ($w_{N=10} - w_{N=100}$); also, $F_i = \frac{\Delta w}{\log N_2/N_1}$

N = number of blows at water content w

C = constant to be determined

The shearing strength of a soil at the plastic limit is a measure of the toughness of the clay; the shearing strength of all soils at the w_L is a constant (very nearly). Thus we can use Eq. (3-4) as an approximate criterion for the relative magnitudes of the shearing resistance of different soils at the plastic limit.

For example, for two soils of identical I_p but different flow indices F_i , if both soils undergo the same change in water content from the w_L , the soil with the steeper curve (largest F_i) will require the fewer number of blows to close the standard groove and thus will have the lowest shear strength for that water content.

Since performing large numbers of liquid-limit tests is time-consuming, the Waterways Experiment Station, Vicksburg, Miss., in Technical Memorandum No. 3-286, June, 1949, analyzed 767 tests and concluded that the liquid limit could be established from a single test using the following equation:¹

$$w_L = w_N \left(\frac{N}{25} \right)^{\tan \beta} \quad (3-5)$$

where w_N = water content at the blow count N of the test

β = slope of the semilog plot of w vs. $\log N$

For this series of liquid-limit values, it was found that $\tan \beta = 0.121$ was a good approximation, resulting in

$$w_L = w_N \left(\frac{N}{25} \right)^{0.121} \quad (3-6)$$

The value of $\tan \beta$ is not 0.121 for all soils; however, good results can generally be obtained from this equation if the single test to obtain w_N is taken for an N count of between 20 and 30. This is so because in this small range of the flow curve, the change in vertical movement (or water content) is small even for steeply sloping curves.

The *plastic limit* has been arbitrarily defined as that water content of the soil at which a thread just crumbles when it is rolled down to a diameter of 3 mm, or approximately 3 mm (refer to Fig. 3-4). This test is somewhat more subjective (operator-dependent) than the liquid-limit test since just what constitutes crumbling and what is a 3-mm. diameter are subject to some interpretation. The diameter can be displayed in the laboratory using wire or welding rods for a visual comparison. With practice, it appears that plastic-limit values can be reproduced to within 1 to 3 percent by different laboratory technicians on the same soil.

Procedure *This Will Be an Individual Project*

A. LIQUID LIMIT (refer to Fig. 3-4)

1. Each person in each group will pulverize a sufficient quantity of the air-dried soil (from the 5 kg put out for drying the previous week) to obtain a representative sample of material passing the No. 40 sieve of 250 ± 10 g. Be sure to discard the residue remaining on the sieve into waste cans as this material is no longer representative of the soil from the field site. Be sure to break the lumps down for sieving; one of the major sources of error is failure to use a truly representative sample, and many of the "fines" are in the lumps.

Do not oven-dry the soil to pass it through the No. 40 sieve as this will reduce the liquid and plastic limits of the soil.

In the interest of student laboratory time, do not presoak the sample prior to performing the test.

2. Next each group will check the height of fall of the liquid-limit machine it will use for a fall of exactly 1 cm (not over ± 0.1 mm). Use the 1-cm calibration block on the end of the grooving tool for making the adjustment. Make the adjust-

¹See Fang (1960). The Washington State Highway Department, working independently, arrived at an equation of this form at approximately the same time.

Figure 3-4

Steps in the liquid-limit test.

- (a) Carefully blended soil. Note uniform, creamy texture.
- (b) Soil placed in liquid-limit cup for test. Note that cup is not nearly full and that front portion only is used.
- (c) Pat of soil grooved with Casagrande grooving tool. Depth is such that tool has only barely trimmed soil pat at deepest part.
- (d) Soil grooved 12.7 mm (visual but scale used for numerical comparison in photograph) while rotating crank at 120 rpm and counting drops of cup onto base of liquid-limit device.
- (e) Water-content sample taken from location of groove closure. Note the quantity in the moisture cup is approximately 45 g. In any case from this area of the soil pat take a water-content sample of at least 40 g.



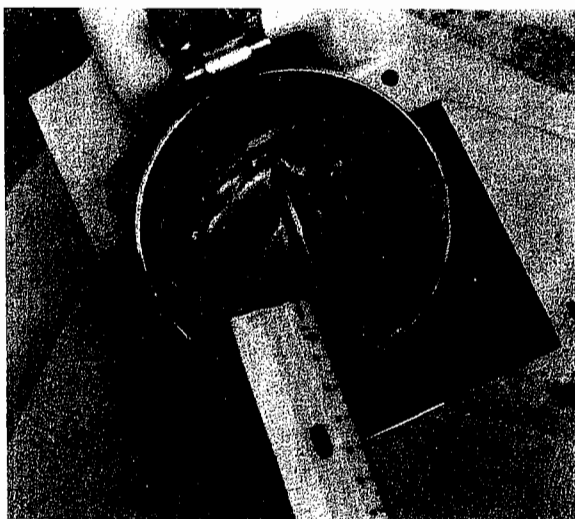
(a)



(b)



(c)



(d)



(e)

ment with respect to the worn spot on the base of the cup, not the shortest fall. If the height of fall is not calibrated within these limits, a water-content error of several percent may be introduced. If the machine is in poor condition or has loose parts, check with the instructor for repair and or replacement.

3. Place the 250 g of soil in a porcelain mixing dish, add a small amount of water, and carefully mix the soil to a uniform color (refer to Fig. 3-4a). Another major source of error is poor mixing of the soil-water mixture. When the color is uniform throughout and the mixture has a creamy appearance, proper mixing has generally been achieved. Continue to add small amounts of water and mix the mass to a uniform color each time. When you have the soil to the point of consistency (stickiness) where you estimate (or by making a trial test) it will take about 50 blows to close the standard groove 12.7 mm, remove about 20 g of the thoroughly mixed soil from the dish for the plastic limit determination. Now add more water and mix to a uniform color until you have the soil to a consistency which will yield a blow count of from 30 to 40 blows to close the standard groove 12.7 mm.
4. Remove the brass cup from the liquid-limit machine and place a small amount of soil to the correct depth of the grooving tool, well centered in the cup with respect to the hinge, and of the general shape shown in Fig. 3-4b. Smooth the surface of the soil pat carefully, and using the grooving tool, cut a clean, straight groove that completely separates the soil pat into two parts. The depth of the soil in the deepest part of the pat should be just even with the top of the ASTM tool (Fig. 3-3b). If the Casagrande tool is used, it should be held perpendicular to the instantaneous tangent to the cup surface and the tool so that the soil pat is not trimmed too shallow in depth (Fig. 3-4c). The soil depth should be just barely trimmed with the shoulders of the tool at the deepest part.

After making the groove, quickly attach the cup to the device and make the blow count. Unnecessary delay will, if the laboratory humidity is low, dry the soil surface, which will affect the blow count. This effect will show when plotting the data by giving an erratic locus of points. Other test errors will also do this, however.

5. Take a moisture sample in the preweighed moisture cups (as large as possible and of about 40 g), being sure to take the water-content sample from the *closed part of the groove* (refer to Fig. 3-4e). Place the lid on the moisture-content container of soil and set it aside temporarily. Remove the remainder of soil from the brass cup and return it to the porcelain dish. Wash and dry the cup.

Add a small amount of water to the porcelain dish of soil, and carefully mix to a uniform color and consistency to yield a blow count of between 25 and 30 + blows. Repeat steps 4 and 5.

6. Repeat the sequence for two additional tests for blow counts of between 20 and 25 and between 15 and 20, for a total of four test determinations.

There should be a difference between the blow counts of the individual trials of at least 2 and preferably 3 or more to yield a reasonably spread plot—but one in which the blow count is not far from 25 blows.

Be sure to clean the brass cup after each test and carefully dry it.

Be sure to have about the same time lapse for each test to eliminate laboratory humidity as a variable.

Do not leave the soil in the brass cup for a long period of time. This would allow adhesion to build up between cup and soil.

7. Weigh the four moisture containers from the tests, *remove the lids*, and place the cups of soil in the oven (110° C) to dry overnight.

It should be obvious now that the addition of water and the blow-count sequence are done in this manner so that the soil is well mixed. It is easier to blend water into the mixture than to blend in new soil if the soil mass becomes too wet

and must be dried to obtain the desired spread of blow counts on each side of 25. It is difficult for the novice to predict the blow count of a soil from a visual inspection, but if he has a blow count of, say, 35 and then water is added, it is reasonable to expect the next test to yield a blow count of less than 35. On the other hand, at an initial blow count of 18, how much dry soil does one add to get a blow count of 22 to 24? The wetter the soil, the more nearly it will act as a viscous liquid.

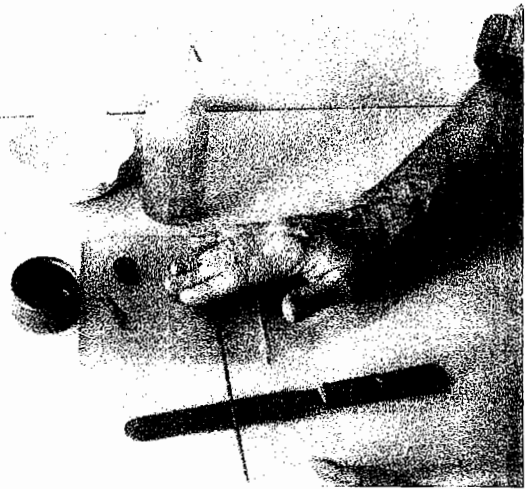


Figure 3-5

Plastic-limit test. Soil is set aside when blow count is at around 50 blows (initial trials). Note use of 3-mm welding rod for visual comparison of the thread diameter. Note extra soil on glass plate for additional trials.

B. PLASTIC LIMIT

The next step is to determine the plastic limit. To increase the precision by eliminating weighing errors, the test should be done as follows:

1. Break the 20- to 30-g "peanut" of soil set aside earlier into several smaller samples.
2. Roll the soil between the fingers and a glass plate or on a piece of paper lying on a smooth surface, with sufficient pressure to form a uniform diameter thread using about 80 to 90 rolling strokes per minute (forward and back = 1 stroke). When the diameter of soil thread becomes 3 mm ($\frac{1}{8}$ in.), break thread into several pieces, re-form into a ball and reroll (Fig. 3-5). Continue this alternative rolling and reballing until the thread crumbles under the pressure of rolling and the soil can no longer be rolled into a thread.

If the thread crumbles at a diameter >3 mm, this is satisfactory to define w_p if the previously rolled thread was to 3 mm. Thread failure may be defined as:

- a. Simply falling apart
- b. With outer tubular (hollow roll) layer that splits from ends inward
- c. Barrel-shaped solid pieces 6 to 8 mm long (for heavy clays)

Do not produce a failure by allowing thread to reach 3 mm by reducing the rate of rolling and/or hand pressure. Exception is with soils approaching nonplasticity, the initial ball may be shaped closer to 3 mm by hand prior to rolling.

3. Repeat this sequence several additional times adding each test to the same moisture can.¹
4. Weigh the covered can, *remove the lid*, and place the can in the oven. Note that you have in effect made the several plastic limit determinations but have eliminated all but one weighing and computation.

¹Some persons prefer to use separate moisture cups for each plastic-limit determination. However, weighing errors tend to offset reproducibility and the method given here is preferred by the author.

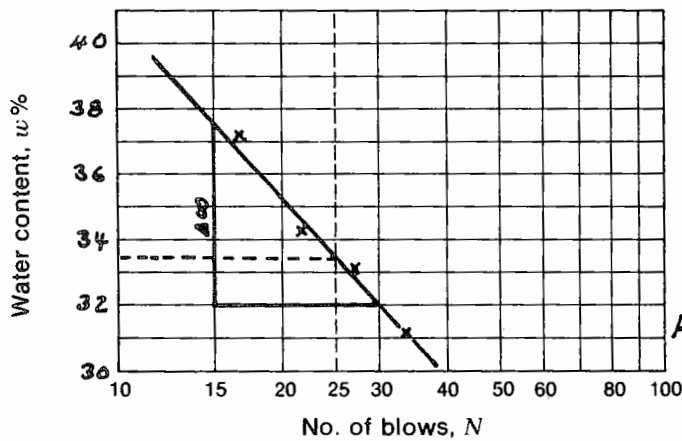
ATTERBERG LIMITS DETERMINATION

Data Sheet 3

Project Exp. No. 3 Job No. —
 Location of Project Bradley Univ Boring No. — Sample No. —
 Description of Soil Brown Silty Clay
 Depth of Sample — Tested By J.E.B. Date 5-4-76

Liquid Limit Determination

Can no.	27	28	31	34		
Wt. of wet soil + can	48.61	55.53	51.71	50.51		
Wt. of dry soil + can	41.19	46.05	42.98	41.54		
Wt. of can	17.33	17.41	17.45	17.36		
Wt. of dry soil	23.86	28.64	25.53	24.18		
Wt. of moisture	7.42	9.48	8.73	8.97		
Water content, $w\%$	31.1	33.1	34.2	37.1		
No. of blows, N	34	27	22	17		



Flow index $F_i = -17.8$

Liquid limit = 33.5%

Plastic limit = 19.4%

Plasticity index $I_p = 14.1$

$$F_i = \frac{\Delta w}{\log N_{c/N_1}} = \frac{32 - 32.6}{\log 21/3} = -17.8$$

Plastic Limit Determination

Can no.	35	37		
Wt. of wet soil + can	23.26	23.03		
Wt. of dry soil + can	22.84	22.64		
Wt. of can	20.63	20.66		
Wt. of dry soil	2.21	1.98		
Wt. of moisture	0.42	0.39		
Water content, $w\% = w_p$	19.1%	19.6		

Figure 3-6
Typical set of liquid- and plastic-limit test data.

Computations

1. Return to the laboratory on the following day and weigh all the dry moisture samples and compute the moisture contents. Plot the liquid-limit data on the graph on the data sheet from data sheet section of this manual and obtain the liquid limit as in Fig. 3-6. Compute the flow index F_i and show on the data sheet page.

Also compute the plastic limit and the plasticity index I_p as

$$I_p = w_L - w_P \quad (3-7)$$

2. Use Eq. (3-6) and compute the liquid limit for each value of N and w_N ; tabulate in your "Discussion" and make a comparison with w_L from the semilog plot.
3. Compute the activity of the soil using the percent finer from the hydrometer analysis for $D = 0.002$ mm if Experiment No. 5 is done in parallel with this experiment. When using Eq. (3-1) for this computation use $C_o = 9$.
4. In your report "Discussion" give six uses for the w_L and w_P data you have just obtained.

Experiment No. 4

SHRINKAGE LIMIT

References

AASHTO T92-68

ASTM D427-61

Chen, F. H., (1975), "Foundations on Expansive Soils," Elsevier Publishing Co., Amsterdam. (Contains numerous recent references.)

Holtz, W. G., and H. J. Gibbs (1956), Engineering Properties of Expansive Clays, *Transactions, ASCE*, vol. 121, pp. 641-677.

Seed, H. B., R. J. Woodward, Jr., and R. Lundgren (1962), Prediction of Swelling Potential of Compacted Clays, *J. Soil Mech. Found. Div., ASCE, SM 3*, June, pp. 53-87.

Objective

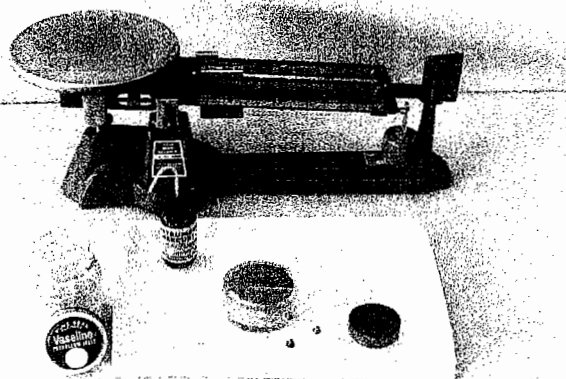
To obtain the moisture content below which no further volume change of a soil mass occurs and to obtain a quantitative indication of the amount of volume change which can occur.

Equipment

Shrinkage dish (refer to Fig. 4-1)
Glass plate with three prongs
Volume dish and flat strike-off plate
Mercury metal
Balance sensitive to 0.1 g

Figure 4-1

Shrinkage-limit equipment. Balance sensitive to 0.1 g (0.01 by estimation); shrinkage dish of monel metal; glass volumetric dish (filled with mercury); circular ground-glass plate to strike of the mercury in glass volumetric dish; glass plate with three prongs (monel metal) used to submerge the dry soil cake into the glass volumetric dish; container of mercury metal; petroleum jelly used to lightly lubricate shrinkage dish so soil cake does not stick and crack. Note use of a handmade shallow cardboard tray to contain any mercury spill. Size is sufficient for ease of taking measurements.



General Discussion

See Experiment No. 3.

Soils which undergo large volume changes with change in water content may be troublesome if used for highway or railroad fills or if structural foundations are placed on them. The soil volume changes can result in bumps in roads and cracks in structures since the volume changes may not, and usually will not, be equal.

The liquid and plastic limits may be used to predict potential trouble in soils due to volume changes. However, to obtain a quantitative indication of how much change in moisture can occur before any appreciable volume change occurs and to obtain, if volume change does occur, an indication of the amount of change, a shrinkage-limit test should be performed.

This test begins with a given volume of *fully saturated soil*, preferably (but this is not absolutely necessary) at a water content above the liquid limit. The soil is dried. It is assumed during drying that down to a certain limiting value of water content, any loss of water is accompanied by a corresponding change in bulk volume (or void ratio). Below this limiting value of water content, no further change in volume occurs with loss of pore water. This limiting value of water content is termed the *shrinkage limit*.

Physically, this means that any moisture changes below the shrinkage limit do not cause soil volume changes. Above the shrinkage limit, volume changes will occur with change in water content. This volume change can be expressed in terms of void ratio and water content, as shown in Fig. 4-2.

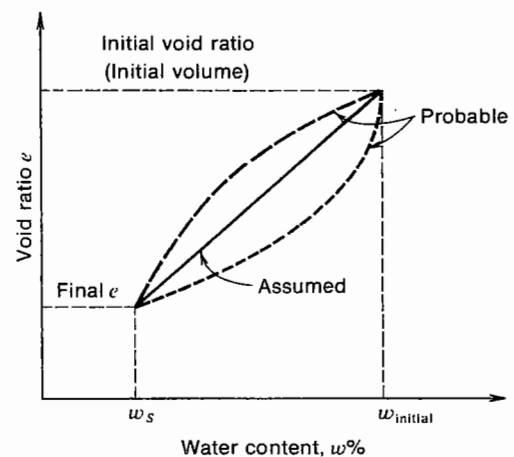


Figure 4-2

Qualitative plot of water content vs. void ratio. Note that dashed lines indicate possible shrinkage paths, which are nonlinear as compared with the heavy-line assumption.

The *shrinkage ratio* gives an indication of how much volume change may occur with changes in water content. The shrinkage ratio is defined as the ratio of the volume change in the soil specimen as a percentage of the dry volume $\Delta V/V_f$ to the corresponding change in moisture content above the shrinkage limit as a percentage of the oven-dry weight of the soil specimen $\Delta w/W_s$. This definition gives the shrinkage ratio SR as the *apparent specific gravity of the dry soil cake*:

$$SR = \frac{\Delta V/V_f}{\Delta w/W_s} = \frac{W_s}{\gamma_w V_f} \quad (4-1)$$

since $\Delta V = \Delta w/\gamma_w$

Symbols in the above and following equations are defined as follows:

- V_f = dry volume of soil (skeleton not solids)
- V_i = initial volume of wet soil at a water content w_i
- ΔV = change in soil volume
- w_i = initial water content of soil
- w_s = shrinkage limit water content
- Δw = change in water content
- γ_w = unit weight of water in consistent units
- W_s = weight of dry soil solids

Linear shrinkage can be calculated from the volumetric shrinkage S_v defined as

$$S_v = (w_i - w_s)SR \quad (4-2)$$

The assumption is made that the reduction in soil volume is uniform in all directions. The linear shrinkage is usually based on the decrease in soil volume as the moisture content changes from some water content (generally the initial value, w_i) to the shrinkage limit, and can be computed as

$$LS = [1 - \sqrt[3]{1.0/(S_v + 1.0)}] \times 100 \quad \text{percent} \quad (4-3)$$

The volumetric shrinkage is used as a decimal quantity in Eq. (4-3).

Sometimes the linear shrinkage is obtained by molding a shrinkage bar of soil in a rectangular mold of, say, $25 \times 25 \times L$ ($= 100$ to 200) mm using the same techniques as for the regular cake-shaped specimens.¹ In this latter case, one can compute the linear shrinkage as

$$LS = \frac{L_o - L_f}{L_o}$$

where L_o = length of wet soil bar
 L_f = length of dry soil bar.

This method of using a soil bar to determine linear shrinkage is not generally considered to be very accurate.

Procedure

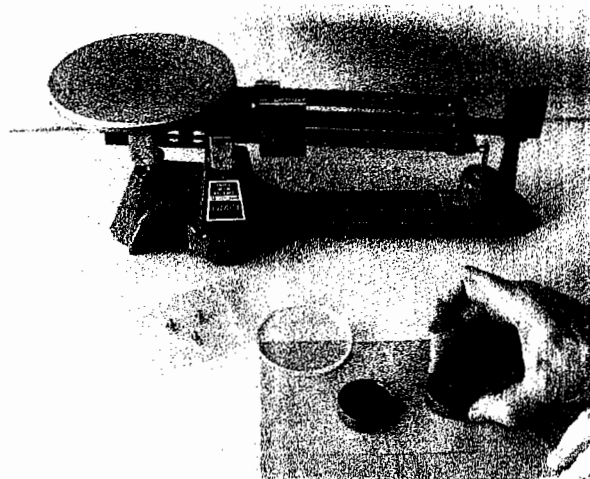
This Will be a Group Project

NOTE: AVOID DIRECT SKIN CONTACT WITH MERCURY AS IT IS A HARMFUL SUBSTANCE.

1. Take about 40 g of the (—) No. 40 sieve² (passing) *representative* material used for the liquid- and plastic-limit tests, and carefully mix with tap (or distilled) water to make a creamy paste which can be placed in the shrinkage dish without any air voids. The soil consistency should be obtained using a water content somewhat above the liquid limit.
2. Lightly coat the inside of the shrinkage dish (either porcelain or monel metal) with petroleum jelly or silicone grease, or spray lightly with Teflon powder (inert substance) to prevent the soil sticking to the dish and/or forming cracks upon drying. Weigh the dish and record the weight.
3. Fill the dish in three layers by placing approximately one-third of the amount of wet soil to fill the dish and tapping it gently on a firm base until the soil flows over the dish and no apparent air bubbles exist. Then repeat with the second and third layers. Strike the dish off smooth using a medium-size spatula, and weigh the dish and wet soil.

¹Several state highway departments have used this linear-shrinkage method instead of the method described specifically for this experiment; the latest AASHTO standard test only standardizes the round soil-cake method outlined in this experiment.

²See sieve designations in Experiment No. 3 and Table 5-1 for standard sieve sizes for areas outside the United States.



(a)



(b)

Figure 4-3

Shrinkage-limit test.

- (a) Filling the shrinkage disk by rapping against table to remove air bubbles. Note an already oven-dried shrinkage sample in shrinkage dish in foreground.
- (b) Preparing to determine volume of dried soil cake by using three-pronged glass plate to submerge cake into glass volumetric cup filled with mercury. Note that cake floats on surface until forced under. Volumetric cup is set in shallow cardboard container to catch mercury overflow. Balance is used to obtain before-and-after weights of glass volumetric cup.

4. Air-dry the wet soil cake in the laboratory until the surface changes to a light color (say, 6 to 8 h). Then oven-dry the dish of soil at 105 to 110°C to constant weight (12 to 18 h). Remove the dried dish of soil from the oven, and obtain the weight of the dish + dry soil. The air drying is to reduce the possibility of the soil cake forming large shrinkage cracks due to too rapid a loss of soil moisture and to avoid losing soil in the oven if the soil "boils."
5. Find the volume of the shrinkage dish in cubic centimeters. This is done by filling the dish to slightly overflowing with mercury. Place the filled dish on a watch glass and press a flat glass plate down on the mercury surface to remove the excess. Return the overflow mercury to the storage container. Pour the mercury from the shrinkage dish into a glass graduate and read the volume of the shrinkage dish directly. Alternatively, weigh the dish + mercury and the empty dish and compute the volume based on the unit weight of the mercury being about 13.53 g/cm³. Thus,

$$V_{\text{dish}} = \frac{\text{weight of mercury}}{13.53} \quad (4-5)$$

The volume of the dish in Eq. (4-5) is the volume of wet soil V_i used in Eq. (4-8).

6. Determine the volume of the dry soil cake V_f by first filling with mercury the glass cup of the shrinkage-limit set (see Fig. 4-1), which has been placed in a

large flat container to catch any mercury spill.¹ Next take the glass plate and press off any excess mercury into the overflow container. Return the excess mercury to the storage flask. Weigh the glass cup filled with mercury and record the weight. Be very careful not to spill mercury as it is nearly impossible to pick up and will mix into and contaminate the soil/dust on the laboratory floor.

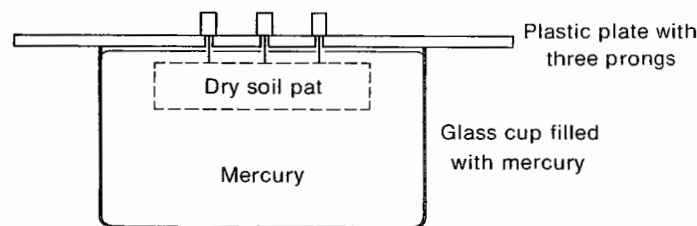
7. With the glass cup still in the flat container, place the dry soil cake on the surface of the mercury (it will float; see Fig. 4-3). By visual observation, siphon off mercury until approximately the volume of the soil cake has been removed. Now take the plate with three prongs, place the prongs on the soil cake, and submerge it, allowing the excess mercury to squeeze out between the plate and the rim of the glass cup (Fig. 4-4). When the glass cup is exactly full with the mercury, soil cake, and the three prongs on the cover plate, remove the cover plate and the soil cake and weigh the glass cup and remaining mercury. The difference between this weight and the weight of the glass cup full of mercury obtained in step 6 is the weight of mercury (Δ_{weight}), displaced by the soil cake. The volume V_f of the dry soil cake is

$$V_f = \frac{\Delta_{\text{weight}}}{13.53} \quad (4-6)$$

Alternatively, place the glass cup in a large dish and catch the overflow when the soil cake is submerged. Place the overflow in a graduated cylinder and obtain the volume directly.

Figure 4-4

Dry soil cake submerged in mercury to determine volume of mercury displaced using glass plate with three prongs. Note prongs do not penetrate soil cake. Even when using cardboard overflow container, siphon mercury from the volumetric cup (after initial weighing) until approximately the volume of the soil cake has been removed, then submerge the cake—this avoids having large quantities of mercury in the container and reduces possibility of skin contact with the mercury.



If the soil cake cracks apart on drying, the test may be redone. For student work, however, one can determine the weight and volume of the largest piece of the cake and by proportion find the volume of the dry soil cake. This requires weighing the entire amount of dry soil as well as the largest piece. Then, by proportion,

$$\frac{V_f}{V_{\text{piece}}} = \frac{\text{total dry weight}}{\text{dry weight of largest piece}} \quad (4-7)$$

8. Compute the shrinkage limit (see Fig. 4-5). The water content at the beginning of the test can be obtained from steps 2 to 4 as

$$w_0 = \frac{W_w}{W_s} 100$$

¹An excellent container can be made using a piece of light cardboard and bending the edges to make a container 1–2 cm deep. Use masking tape to hold the edges (see Fig. 4-1).

SHRINKAGE LIMIT TEST

Data Sheet 4

Project Shrinkage Limit Job No. _____Location of Project Bradley University Boring No. _____ Sample No. _____Description of Soil Dk. Red-Brown Clay Depth of Sample _____Tested By J.E.B. Date of Testing 3-17-76Wt. of coated dish + wet soil = 37.83 gWt. of coated dish + dry soil = 31.92 gWt. of coated dish = 10.43 gWt. of soil, W_s = 21.49 gWt. of Water, W_w = 5.91 gWater content, $w_o\%$ = 27.50 %Vol. of Wet Soil, V_o = 14.95 cm³ (Step 5 of procedure: Vol. of shrinkage dish = V_o).Vol. of Dry Soil, V_f = 11.61 cm³ (Step 7 of procedure)Shrinkage limit, $w_s = w_o - \frac{(V_o - V_f) \gamma_w}{W_s} \times 100 = \underline{12.0 \%}$ Shrinkage ratio, $SR = W_s/V_f = \underline{21.49/11.61 = 1.85}$ *Wt of Shrinkage Dish = 21.22 g**Wt of Shrinkage Dish + Hg = 616.73 g**Wt of Shrinkage Dish + Hg after Submery. Soil Cake = 459.62 g**Wt of Hg. Displaced = 616.73 - 459.62 = 157.11 g*

$$V_f = \frac{157.11}{13.53} = \underline{11.61 \text{ cm}^3}$$

*Wt of Soil Cake Dish = 10.42 g**Wt of Soil Cake Dish + Hg = 212.65 g*

$$V_o = \frac{212.65 - 10.42}{13.53} = \underline{14.95 \text{ cm}^3}$$

$$w_s = 27.50 - \frac{(14.95 - 11.61)(1)(100)}{21.49} = 27.50 - 15.54 = 11.96$$

(12.0)

Figure 4-5

Typical shrinkage-limit test data and computations.

The shrinkage limit is defined as the water content below which no further volume change occurs. Since one starts with a fully saturated soil condition and it is assumed the soil remains saturated to the shrinkage limit, the expression for the shrinkage limit is derived as follows.

The change in soil volume is

$$\Delta V = V_i - V_f \text{ (from steps 5 and 7)} \quad (4-8)$$

The water content at the shrinkage limit is computed from the initial water content and the change in water content to the shrinkage limit as

$$w_s = w_i - \Delta w$$

but the change in volumetric water content (and using the definition $w_w = wW_s$) is

$$\frac{\Delta w W_s}{\gamma_w} = \Delta V$$

and solving for Δw , obtain:

$$\Delta w = \frac{\Delta V \gamma_w}{W_s}$$

Substituting into Eq. (4-8) and in cgs units, $\gamma_w = 1$, obtain the shrinkage limit as

$$w_s = w_i - \frac{\Delta V}{W_s} 100 \quad (4-9)$$

9. Compute the shrinkage ratio SR using Eq. (4-1).
10. Compute the linear shrinkage using Eq. (4-3).

The Report

In preparing the report for this project, discuss why mercury is used in steps 5 and 7. The sample computation sheet should include the derivation of Eq. (4-1).

Plot the w_L and I_p of this soil on the A chart of Fig. 8-2 and extend a line to the intersection point of the UL and A lines. Where this laboratory line crosses the w_L axis obtain the water content and compare with w_s . Also using an approximate average slope of the UL and A lines, extend this projection from the w_L, I_p coordinates and obtain the water content intercept on the w_L axis. Which approximation gives the best shrinkage-limit value? Could this method be used to eliminate the shrinkage-limit test? How may you determine this possibility without personally performing a large number of w_s tests?

Experiment No. 5

GRAIN SIZE ANALYSIS—MECHANICAL METHOD

References

AASHTO T87-70 (Sample Preparation); AASHTO T88-70 (Test Procedures)
ASTM D421-58 and D422-63

Objective (Instructor¹ see note at end of "Procedure")

To introduce the student to the method of making a mechanical grain-size analysis of a soil and presenting the resulting data.

Equipment

Set of sieves (see typical list in "General Discussion"; also Fig. 5-1 and Table 5-1)
Mortar and pestle or a mechanical soil pulverizer
Balance sensitive to 0.1 g

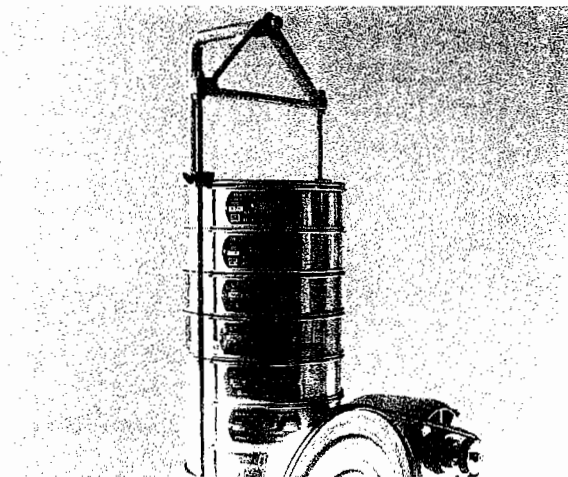


Figure 5-1
Typical stack of sieves in a mechanical sieve shaker.

General Discussion

Some type of grain-size analysis is universally used in the engineering classification of soils (see Experiment No. 8). Part of the suitability criteria of soils for road, airfield, levee, dam, and other embankment construction is the grain-size analysis. Information obtained from the grain-size analysis can be used to predict soil-water movement, although the permeability tests (Experiments No. 11 and 12) are more generally used. The susceptibility to frost action in soil, an extremely important consideration in colder climates, can be predicted from the grain-size analysis.

Very fine soil particles are easily carried in suspension by percolating soil water and underdrainage systems will rapidly fill with sediments unless they are

¹It is suggested that a stock sand (using concrete sand with some fines contamination) be used in this experiment so that not more than 10 percent passes the No. 200 sieve so that C_u and C_c may be computed. If stock sand is used, omit the washing sequence. Use the cohesive soil of Experiment No. 3 in Experiments No. 6, 7, 9, and 14.

Table 5-1 Standard sieve sizes. All U.S. sieves are available in 20-cm, and most in 30.5-cm, diameter. The current U.S. designation uses 100 down to 6.3 mm as a size designation; from No. 4 (4.75 mm) to No. 18 (1.00 mm), the mesh opening is in millimeters; from No. 20 to No. 400, the mesh opening is in micrometers ($\mu\text{m} = 0.001 \text{ mm}$). The No. 20 sieve is $850 \mu\text{m}$ or 0.850 mm . From the No. 4 sieve and smaller, the size is approximately $N_i \approx (N - 1)/\sqrt{2}$ (Example: No. 5 = $4.00 \text{ mm} = 4.75/\sqrt{2}$)

U.S. ^a		BRITISH STANDARD ^b		FRENCH ^c		GERMAN DIN ^d	
Size or No.	Opening (mm or μm)	No.	Opening (mm)	No.	Opening (mm)	Designation (μm)	Opening (mm)
	100.0 mm						
	75.0						
	63.0						
	50.0						
	45.0						
	37.5						
	31.5						
	25.0						25.0
							20.0
	19.0						
							18.0
	16.0						16.0
	12.5						12.5
							10.0
	9.5						
	8.0						8.0
	6.3						6.3
				38 ^e	5.000		5.0
4"	4.75						
5	4.00			37	4.000		4.0
6	3.35	5"	3.353				
				36	3.150		3.150
7	2.80	6	2.812	35	2.500		2.500
		7	2.411				
8	2.36						
10	2.00	8	2.057	34	2.000		2.000
12	1.70	10	1.676	33	1.600		1.600
14	1.40	12	1.405	32	1.250		1.250
		14	1.204				
16	1.18						
18	1.00	16	1.003	31	1.000		1.000
20	850 μm	18	.853	30	.800	800	.800
25	710	22	.699	29	.630	630	.630
30	600	25	.599				
35	500	30	.500	28	.500	500	.500
40 ^f	425	36 ^f	.422				
45	355	44	.353	27 ^f	.400	400 ^f	.400
50	300	52	.295	26	.315	315	.315

U.S. ^a		BRITISH STANDARD ^b		FRENCH ^c		GERMAN DIN ^d	
Size or No.	Opening (mm or μm)	No.	Opening (mm)	No.	Opening (mm)	Designation (μm)	Opening (mm)
60.	250	60	.251	25	.250	250	.250
70	212	72	.211	24	.200	200	.200
80	180	85	.178	23	.160	160	.160
100	150	100	.152				
120	125	120	.124	22	.125	125	.125
140	106	150	.104	21	.100	100	.100
170	90	170	.089	20	.080	90	.090
200	75	200	.076			80	.080
						71	.071
230	63	240	.066	19	.063	63	.063
						56	.056
270	53	300	.053	18	.050	50	.050
325	45					45	.045
400	38			17	.040	40	.040

^aASTM E-11-70 (Part 41).

^bBritish Standards Institution, London BS-410.

^cFrench Standard Specifications, AFNOR X-11-501.

^dGerman Standard Specification, DIN 4188.

^eFor standard compaction test.

^fFor Atterberg limits.

properly surrounded by appropriately graded granular materials. The proper gradation of this material, termed *filters*, can be predicted from the grain-size analysis.

The grain-size analysis is an attempt to determine the relative proportions of the different grain sizes which make up a given soil mass. Obviously, to have significance the sample must be *statistically representative* of the soil mass. Actually it is not possible to determine the individual soil sizes—the test can only bracket the various ranges of sizes. This is accomplished by obtaining the quantity of material passing through a given sieve opening but retained on a sieve of smaller sized openings and then relating this retained quantity to the total sample. It is evident that the material retained on any sieve in this manner consists of particles of many sizes, all of which are smaller than the openings of the sieve through which the material passed but larger than the openings of the sieve on which the soil is retained.

The sieves are made of woven wire with rectangular openings ranging in size from 101.6 mm (4 in) in the coarse series to the No. 400 (0.038 mm) in the fine series. Table 5-1 lists generally available sieves together with the corresponding mesh openings. The No. 200 sieve (0.075 mm) is the smallest practical sieve size. This mesh is about the finest size which will permit a relatively free passage of water. Soil, of course, provides considerably more resistance than water to sieving; thus, sieve sizes smaller than No. 200 are more academic than practical (see Fig. 5-2).

Table 5-1 gives the latest ASTM and U.S. National Bureau of Standards official sieve designations. These sizes are slightly different from former (prior to 1970) designations and/or those that may be found on older sieves still in service, i.e., typically

Pre-1970	Current
4-in (101.6 mm)	100 mm
1½-in (38.1 mm)	37.5
¾-in (6.35 mm)	6.3
No. 20 (0.841 mm)	0.850
No. 100 (0.149 mm)	0.150
No. 200 (0.074 mm)	0.075

The reader should use the current size designations (as in Table 5-1) for recording and plotting data since

1. There is little-to-no improved precision to be gained, and
2. The older sieves will rapidly disappear as they wear out and are replaced.

All the soil classification systems use the No. 200 sieve as a dividing point; i.e., classifications are in terms of the amount retained or passing the No. 200 sieve. Occasionally it is desired to know the *approximate* range of grain sizes smaller than the No. 200 sieve. When this need arises (as it does in most university laboratories for student familiarization with the procedure), the method of the next project, "Grain-size Analysis—Hydrometer Method," is used.

The sieving process does not provide information on the shape of the soil grains, i.e., whether they are angular or rounded. It only yields information on grains which can pass, or with proper orientation do pass, through rectangular sieve opening of a certain size. Obviously, not all grains in larger samples which can pass a given opening do pass, since they may not ever become properly oriented with the square opening. The smaller particles may not be broken down to the elemental particle size in the pulverizing process, or the finer particles—especially the (–) No. 200 sieve size (i.e., material which will pass the No. 200 sieve)—may adhere to the larger particles as dust and not pass the proper sieve openings.

Information obtained from the grain-size analysis is presented in the form of a curve. In order to compare soils and to visualize more easily the distribution of the grain sizes, and since a typical fine-grained soil mass may have particles ranging from 2.00 mm or larger to at least 0.075 mm (No. 200 sieve), which would require a very large scale to give the same weight and reading precision to all the sizes, it is customary to plot grain sizes on a logarithmic scale. Standard procedure uses the *percent passing* (also termed *percent finer*) as the ordinate plotted to a natural scale of the grain-size distribution curve. Some literature displays the grain-size curve plotted with increasing grain size from left to right. This gives the same shape curve as obtained by turning the author's plot over and looking at the back.

Typical grain-size distribution curves for *fine-grained soils*¹ are shown in Fig. 5-4. One of the curves is obtained from the data shown in Fig. 5-3. A distribution for a coarser soil would be shifted to the left.

It should be evident that a grain-size distribution curve can only be approximate. This is for the several reasons considered here, including physical limitations on obtaining a statistically representative sample, the presence of soil lumps, the practical limitations of using sieve mesh openings for irregular-shaped soil particles, and the limit on the number of sieves used in a "stack" for the analysis. The accuracy of analysis is more questionable for fine-grained soils (finer than the No. 4 sieve) than for the coarser soils, and the common, but widely used, practice of using oven-dry soils may influence the analysis somewhat. The author

¹Fine-grained is used here in the context that 90 to 95 percent passes the No. 4 (sometimes the No. 10) sieve.

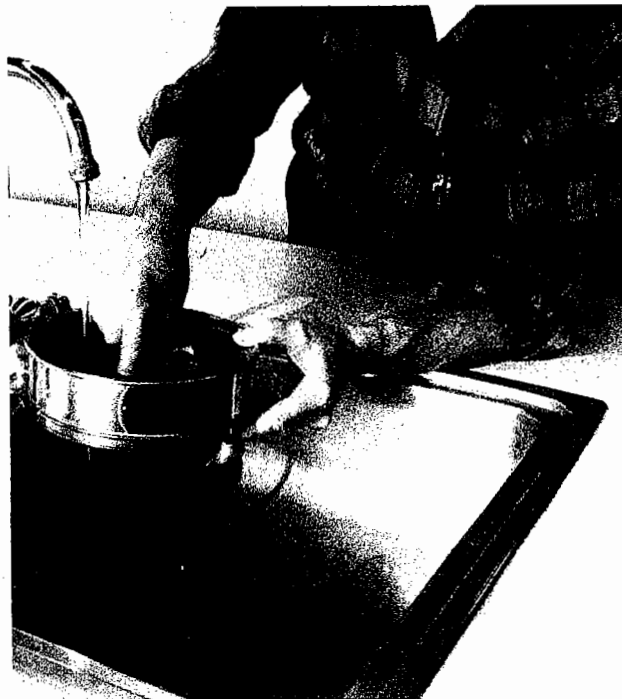


Figure 5-2

Washing material through a No. 200 sieve. Deeper sieves are available especially for this purpose; however, with care the regular No. 200 sieve can be used as shown here.

is of the opinion that oven drying is one of the least significant test factors; a representative sample and the reduction of lumps to elemental particles via sieve washing are of far more importance. The grain-size distribution curve obtained from the following "Procedure" is satisfactory for predicting the behavior of cohesionless soils and to obtain the relative quantities of (+) and (-) No. 200 material for soil classification.

The author has found that the most reliable and most easily reproducible method of performing the sieve analysis on a fine-grained soil with more than 4 or 5 percent passing the No. 200 sieve is to take an oven-dry quantity of material, break it as fine as possible, then wash it on the No. 200 sieve (as illustrated in Fig. 5-2), oven-dry the residue, and sieve through a stack consisting of 5 to 6 sieves with a range of sizes to provide a reasonable range of plotted points. This method ensures that very little dust will adhere to the larger particles and that the reducible lumps are water-softened, the clay particles washed through the sieve, and upon drying the elemental particles remain separated. This method is used to obtain curve A (data not shown) of Fig. 5-4. This is a particularly desirable method where a microscope is used to determine the grain shape as a supplement to the grain-distribution curve.

Sieve washing is not desirable or practical for soils (gravels and gravelly sands) when less than 10 to 15 percent passes the No. 10 (2.00-mm) sieve. Sieve washing is usually not necessary when only 5-10 percent passes the No. 100 (0.150-mm) sieve for the fine-grained soils. In any case, the larger samples necessary with coarse-grained soils (1,000-5,000 g) makes washing impractical.

Due to the statistical distribution of particle sizes even the most representative soil samples do not yield reproducible distribution curves. One has obtained a "reproducible" analysis if the percent finer of any size is on the order of ± 1.5 percent for (-) No. 4 soil and perhaps ± 2.5 percent for coarser grained soils. This is why the scale of the enclosed semilog graph data sheet is satisfactory for general use.

A sieve stack generally consists in a series (usually 6 or 7) sieves with the sizes approximately doubling in opening from the bottom to top sieve as 6, 12, 24 mm or

0.075, 0.150, 0.300 mm, etc. For convenience and practical reasons such as sieve availability, size of stack, introducing control sieves as No. 4, No. 100, No. 200, or other size, some sizes may be omitted; e.g., No. 10, 30, 50, 100, 200, instead of No. 10, 20, 40, 80, 160, etc. This is acceptable since strict adherence to the doubling ratio does not greatly improve the distribution curve and only enough sieves need be used to produce the curve reasonably and with statistical reliability. The doubling ratio should be remembered and used as a guide in developing the sieve stack. A No. 200 sieve is always used as the bottom sieve for fine-grained soils; a No. 100 sieve is always used as the bottom sieve for concrete sand.

From the grain-size distribution curve, grain sizes such as D_{10} , D_{85} , D_{60} , etc., can be obtained. The D refers to the grain size, or apparent diameter, of the soil particles and the subscript (10, 85, 60) denotes the percent which is smaller. For example, $D_{10} = 0.15$ mm from curve B of Fig. 5-4 means that 10 percent of the sample grains are smaller than 0.15 mm. The D_{10} size is also called the *effective size* for a soil.

An indication of the spread (or range) of grain sizes is given by the *coefficient of uniformity* C_u , defined as

$$C_u = \frac{D_{60}}{D_{10}} \quad (5-1)$$

A large value of C_u indicates that the D_{60} and D_{10} sizes differ appreciably. It does not ensure that a condition of gap grading, as when sizes are missing or present in very small relative quantities, does not exist. The *coefficient of concavity* C_c is a measure of the shape of the curve between the D_{60} and D_{10} grain sizes, defined as

$$C_c = \frac{D_{30}^2}{(D_{10}D_{60})} \quad (5-2)$$

Values of C_c greatly different from 1.0 indicates grain sizes missing between the D_{60} and D_{10} sizes. Typical computations for C_u and C_c are shown on Fig. 5-4 for soil B . Note that values computed to 0.1 are sufficiently precise as shown. These values are used in soil classification in Experiment No. 8. The D_{15} and D_{85} sizes may be used in determining suitability of the soil for filter design in an earth dam or to surround the openings of a pipe laid in the ground for a subdrainage system.

The method of performing the grain-size distribution analysis in this laboratory project is essentially that used by the U.S. Corps of Engineers and most consulting firms, but differs from that suggested by ASTM which divides the *air-dry* soil into (+) and (−) No. 10 (2.00 mm) size material. The quantity of (+) No. 10 material depends on the maximum size of particles and ranges from 500 to 5000 g somewhat as shown on Fig. 5-3. Next the (+) No. 10 material is sieved as described here (but without washing) through a sieve stack based on inspection of the particle range and use of an approximate doubling ratio previously described. The (−) No. 10 material is analyzed as in Experiment No. 6 and at the end of the test is then washed through the No. 200 sieve, the residue oven-dried, and a sieve analysis performed. The distribution curve obtained is a composite of the (+) No. 10, hydrometer and sieve analysis of the oven-dried residue. The author does not recommend this procedure (done primarily to avoid oven drying) because a minimum of 115 g of (−) No. 10 material is used, which is more than the capacity of the 152H hydrometer. Additionally, many of the particle sizes (2.00 to whatever, mm) will be too large for the hydrometer theory; the method is less straightforward, and lastly, oven drying is not that significant a factor for the usual use of the grain-size curve. Should oven drying be a particular factor, of course, the laboratory work should be appropriately modified.

For soils where more than 10 to 12 percent passes the No. 200 sieve and a hydrometer analysis is also to be performed, the reader should study both the "Procedure" following, and Experiment No. 6 including, in particular, Method B.

Procedure *This Should Be a Group Project*

1. Each group will obtain exactly 500 g (200 g at discretion of instructor, but larger samples are statistically more representative) of oven-dry soil from the bag of stock material obtained from the field in Experiment No. 2 or from the container of soil supplied by the instructor for this experiment. Be sure the sample is *representative* by using a sample splitter (if available) or by random spooning of the soil from the container and regularly remixing the remaining stock until you obtain the necessary quantity.

If the sample is to be washed, it is not necessary to pulverize the soil but the washing process is considerably speeded by first pulverizing and dry sieving as much as practical through the No. 200 sieve and discarding that passing.

2. If the sample contains appreciable gravel, very few fines, or at the discretion of the instructor washing is to be omitted, go to step 4.

Place the test sample on the No. 200 sieve and carefully wash the material through the sieve using tap water (see Fig. 5-2) until the water is clear. Be very careful in this process not to damage the sieve or lose any soil by splashing the material out of the sieve.

3. Carefully pour the residue, using backwashing into a large weighed dish, and let it set for a short period of time until the top of the suspension becomes clear. Pour off as much of the clear top water as possible, then place the dish and remaining soil-water suspension in the oven for drying.
4. On the following day, return to the laboratory and weigh the oven-dry residue (Omit, of course, if you do not wash).

Either immediately (if not using washing) or the following day (with washing) run your sample through a stack of sieves varying from larger sizes to smaller sizes from top down. Since the objective of the project is a semilog plot of grain size versus percent finer, it will be necessary to obtain a reasonable distribution of points across the potential range of soil sizes present.

For sandy to fine-grained soils, two recommended sieve stacks (in the order from top to bottom) are as follows:

TYPICAL SIEVE STACK		ALTERNATIVE SIEVE STACK	
Sieve No.	Opening, mm	Sieve No.	Opening, mm
Lid		Lid	
4	4.75	4	
10	2.00	10	
20	0.850	30	0.600
40	0.425	50	0.300
60	0.250	100	0.150
140	0.106	200	0.075
200	0.075		
Pan		Pan	

If small gravel is present a 12.5 and/or 6.30 mm sieve may be placed before the No. 4 sieve in the stack. For large-sized gravelly samples, larger sample

weights should be used in an appropriate stack of sieves. A typical stack might be as follows (depending on maximum size of aggregate):

- 50 mm (depending on maximum size of aggregate)
- 25 mm using visual inspection)
- 12.5 mm
- 6.3 mm
- 2.00 (No. 10)
- 0.85 (No. 20)
- 0.150 (No. 100)

Sample weights should range from 1500 g for 19 mm gravel to 5000 g for 75 mm maximum size material.

5. Place the stack of sieves in a mechanical sieve shaker (if available) and sieve for 5 to 10 min, depending on an initial visual inspection of the probable difficulty involved and quantity of material. Note that if the entire stack of sieves will not fit into the mechanical shaker, perform a shaking operation by hand until the top few sieves can be removed from the stack; place the remainder of the stack in the mechanical shaker. If a mechanical shaker is not available, shake by hand for about 10 min. Alternate the mode of shaking so that the grains are being continually moved across the sieve screens—do not shake in a defined pattern.
6. Remove the stack of sieves from the shaker and obtain the weight of material remaining on each sieve. Sum these weights and compare with the weight obtained in step 4 (the oven-dry residue you started with). This is to detect any loss of soil in the mechanical sieving operation. A loss of more than 2 percent by weight of the residue weight (step 4) is considered unsatisfactory and the test should be repeated. *Question:* What might account for the discrepancy if you have a larger weight than you started with?
7. Compute the percent retained on each sieve by dividing the weight retained on each sieve by the *original sample weight* used in step 2. This is valid since any material passing the No. 200 sieve will pass any sieve above it in the stack.
8. Compute the percent passing (or percent finer) by starting with 100 percent and subtracting the percent retained on each sieve as a cumulative procedure. For example, from the displayed sieve-analysis data shown in Fig. 5-3, the quantity of 490.3 g ($500 - 9.7$ g) passed the No. 4 sieve; the percent retained is $(9.7/500) \times 100 = 1.9$ percent and the percent passing must be that not retained, or $100 - 1.9 = 98.1$ percent. On the No. 10 sieve, 39.5 g, or 7.9 percent was retained. Since everything not retained was passed, the percent passing is

$$98.1 - 7.9 = 90.2 \text{ percent}$$

Note that we used 98.1 here since that was the percent that arrived to the No. 10 sieve. In general, the percent passing is computed as

$$\text{Percent passing} = \text{percent arriving} - \text{percent retained} \quad (5-3)$$

9. Each individual should make a semilogarithmic plot of grain size versus percent finer using the graph on the data sheet for this experiment (see Fig. 5-4 and obtain data sheet from data sheet section of manual).
 - a. If less than 12 percent passes the No. 200 sieve, compute C_u and C_c and show on your graph as in Fig. 5-4 (also show in your "Discussion"). Answer the question: Why is C_u and C_c computed only if less than 12 percent passes the No. 200 sieve?

Data Sheet 5

Soil Sample Size (ASTM D1140-54)

Wt. of dry sample + container	893.7
Wt. of container	421.2
Wt. of dry sample, W_s	472.5 g

$$500 - 472.5 = 27.5 \text{ g washed}$$
[illegible]

Figure 5-3
Typical mechanical analysis data (used to construct curve *B* of Fig. 5-4).

GRAIN SIZE DISTRIBUTION

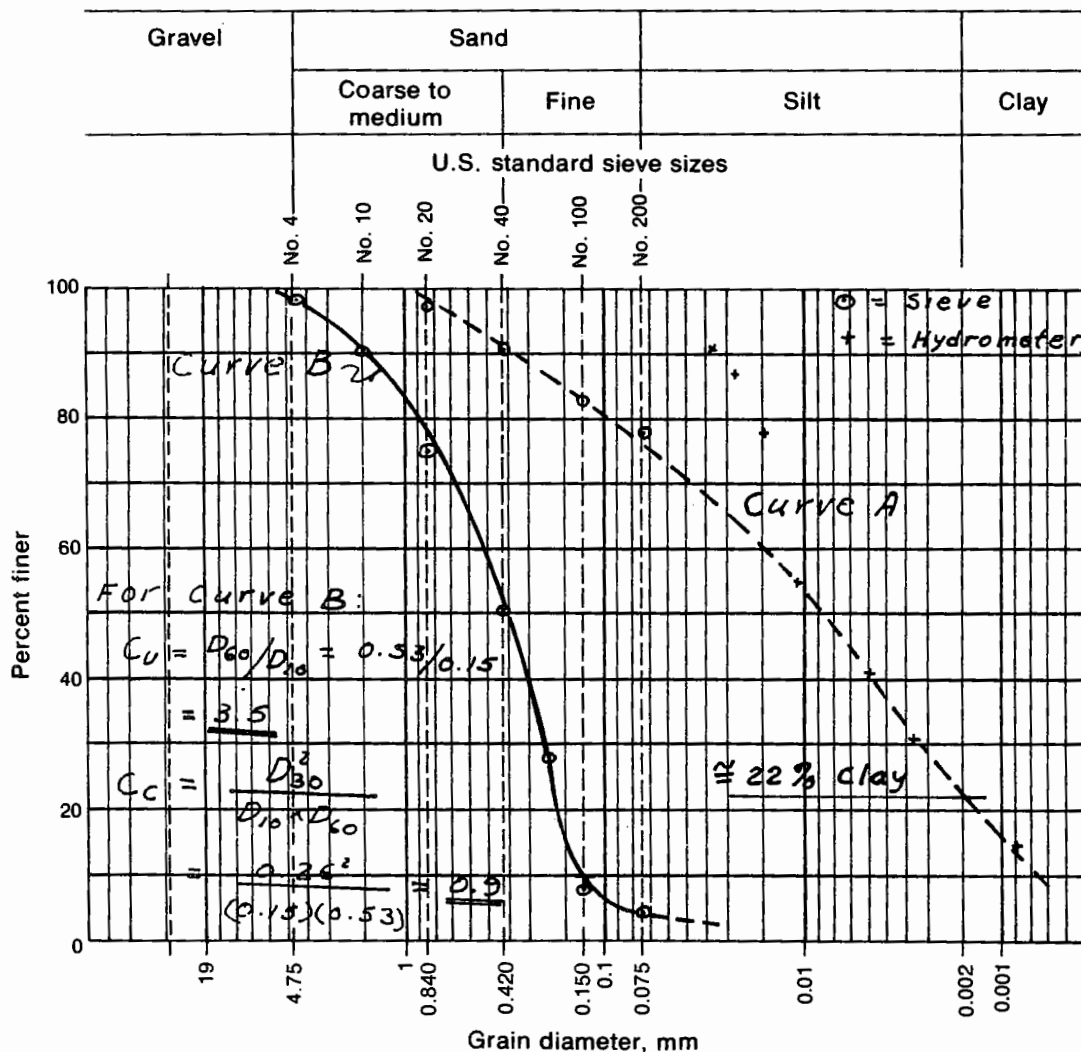
Data Sheet 6

Project Illustrate Sieve Analysis Job. No. _____

Location of Project Bradley Univ. Boring No. _____ Sample No. _____

Description of Soil Sand & Clay Depth of Sample _____

Tested By. JEB Date of Testing Sand: 7-12-76



Visual soil description Soil B: Brown, Medium Coarse, Clean Sand

Soil classification: Note: Soil A from Exp. 6 but not all points plotted

System _____

Figure 5-4

Typical grain-size curves. Curve B is constructed from Fig. 5-3. The hydrometer analysis part of curve A is from Fig. 6-4. Note the discontinuity in curve A between the mechanical and hydrometer data; major weight is given the mechanical data.

- b. If more than 12 percent of your sample passes the No. 200 sieve and a hydrometer analysis (Exp. No. 6) is to be performed on this soil hold your data until Exp. No. 6 is completed so that your grain size curve can be completed before submitting your report.
10. In your report "Discussion" give 5 uses for the grain size distribution curve which has just been made.

Note to Instructor

If you used a coarse, granular soil with less than 10 percent passing the No. 200 sieve so that the students can compute C_u and C_c and omit the washing sequence, it is suggested that the students also wash a 200-g sample of the soil to be used in Experiment No. 6 through the No. 200 sieve during this laboratory and determine the percent passing various sieves. Later this data can be combined with the hydrometer analysis of the next lab period.

Experiment No. 6

GRAIN SIZE ANALYSIS—HYDROMETER METHOD

References

AASHTO T87-70 and T-88-70
ASTM D421-58 and D422-63

Objective

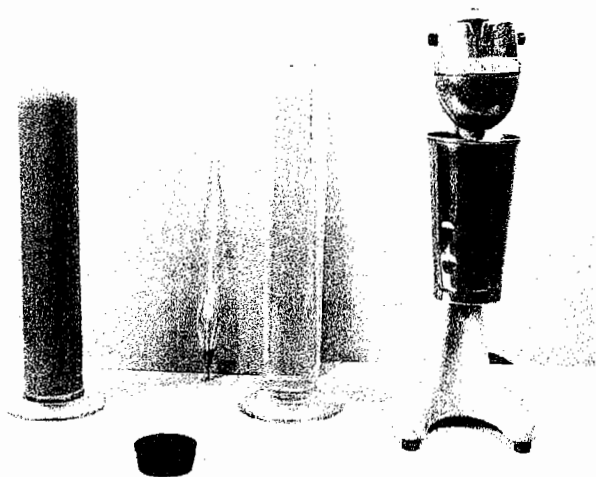
To familiarize the student with a method of obtaining the *approximate* grain-size distribution of soils where appreciable quantities of the soil pass the No. 200 sieve.

Equipment

Sedimentation cylinder (1000 cm³ cylinder), also termed a *hydrometer jar* (Fig. 6-1)
Hydrometer (152H model preferably)
Soil-dispersion device (malt mixer)
Dispersion agent [sodium hexa-metaphosphate (NaPO₃), trade name Calgon, or sodium silicate (Na₂SiO₃), also called *water glass*]
Hydrometer jar bath (optional)
Thermometer

Figure 6-1

Equipment for hydrometer analysis: Hydrometer jar on left filled to 1000-ml mark with soil-water suspension; 1000-ml graduated cylinder on right is used as control jar; malt mixer is used to thoroughly mix soil-water suspension; large rubber stopper (No. 12) for final mixing of soil-water suspension; hydrometer (152H) in background; thermometer suspended in control jar using semistraightened paper clip.



General Discussion

The hydrometer analysis is a widely used method of obtaining an estimate of the distribution of soil particle sizes from the No. 200 (0.075 mm) sieve to around 0.001 mm. The data is plotted on a semilog plot of percent finer vs. grain diameters and may be combined with the data from a mechanical analysis of the material retained (+) on the No. 200 (or other size) sieve as in Experiment No. 5. A combined curve is shown as curve A of Fig. 5-4.

The principal value of the hydrometer analysis appears to be obtaining the percent clay (percent finer than 0.002 mm) since the grain-size distribution curve

when more than 12 percent is (—) No. 200 is not used in any soil classification system and there is no particular soil behavior depending on any intrinsic curve shape. Soil behavior for the cohesive soil fraction depends principally on the type and percent of clay mineral, geologic history, and water content rather than on the distribution of particle sizes.

The hydrometer analysis utilizes the relationship among the velocity of fall of spheres in a fluid, the diameter of the sphere, the specific weights of the sphere and of the fluid, and the viscosity of the fluid as expressed by the English physicist G. G. Stokes (ca. 1850) in the equation termed *Stokes' law*:

$$v = \frac{2\gamma_s - \gamma_f}{9\eta} \left(\frac{D}{2}\right)^2 \quad (6-1)$$

where v = velocity of fall of the spheres, cm/s

γ_s = specific weight of the sphere (specific weight = density $\times g$ = mass/unit volume \times gravity = g/cm³ in cgs system)

γ_f = specific weight of fluid (usually water and see Table 6-1)

η = absolute, or dynamic, viscosity of the fluid, dyne·s/cm² (or g/cm·s)

D = diameter of sphere, cm

g = 980.7 cm/s²

1 g = 980.7 dynes

Solving Eq. (6-1) for D and using the specific weight of water, γ'_w , we obtain

$$D = \frac{18\eta v}{\gamma_s - \gamma'_w} \quad \text{cm} \quad (6-2)$$

The range of soil particle diameters D for this equation to be valid is approximately

$$0.0002 \text{ mm} \leq D \leq 0.2 \text{ mm}$$

since larger grains cause excessive fluid turbulence and very small grains are subject to Brownian movement (i.e., subject to particle forces of attraction and repulsion).

Obviously, to solve Eq. (6-2) one needs to obtain the velocity term v , to know the correct values of γ_s and γ'_w , and to have access to tables of viscosity of water. Since the specific weight of water and its viscosity vary with temperature, it is evident that this variable must also be reckoned with.

To obtain the velocity of fall of the particles, the hydrometer is used. This is a device originally developed to read the specific gravity of a solution, but by altering the scale it can be made to read other values.

By mixing a given quantity of soil with water and a small amount of a dispersing agent to form a 1000 cm³ quantity of solution, one obtains a solution with a specific gravity larger than 1.000 (since the G of distilled water is 1.000 at 4°C). The dispersing (also deflocculating) agent is added to neutralize the charges on the smaller soil grains, which often have plus or minus charges. With proper orientation, these charged soil grains will be attracted to each other with sufficient force to remain stuck together, thus creating larger particles. According to Stokes' law, these larger particles will settle faster through the fluid than the smaller particles. Sodium hexa-metaphosphate, also called sodium metaphosphate (NaPO₃), and sodium silicate, or water glass (Na₂SiO₃), are two materials often used as dispersal agents to neutralize the soil-particle charges. The exact amount and type of dispersing agent required will depend on the soil and may have to be established by trial. A 125 cm³ quantity of 4 % solution of sodium metaphosphate in the 1000 cm³ of soil-water suspension has usually been found to be adequate. If the soil-water suspension clears in a short time—in, say, 2 to 3 h—the amount of

dispersing agent should be increased or a different type used, as a soil with any appreciable quantity of clay-sized particles would remain turbid (muddy) for several days.

Sodium metaphosphate yields an acidic solution (turns blue litmus paper pink) and thus would be expected to be more efficient as a dispersing agent in alkaline soils. Sodium silicate, on the other hand, yields an alkaline solution (turns pink litmus paper blue) and should be more efficient in acidic soils or soils whose pH is less than 7. Most clay soils will be "alkaline." The presence of certain salts or other impurities, however, may create an "acidic" condition. Strictly a sample soil-water solution should be tested for pH prior to arbitrarily using a dispersal agent.

It may be noted that this test as performed by some soil engineers in England involves a bit more refinement: About 100 g of soil is first treated with about 100 ml of 6% hydrogen peroxide solution until gas bubbles are no longer given off (this removes organic materials). This treatment may take several days and require additional hydrogen peroxide. Next, the soil is treated with about 100 ml of 0.2 N hydrochloric acid (17 ml of concentrated HCl acid per 1000 cm³ of H₂O makes a 0.2 N solution) to remove any calcium compounds which might decompose during the test. The treatment is continued until the soil gives an acid reaction to litmus (blue litmus turns pink). The soil is now ready for use, and about 50 g of treated damp soil is mixed with 250 ml of solution containing 8 g of sodium oxalate (Na₂C₂O₄) per 1000 cm³ as a dispersing agent. Sodium oxalate yields an alkaline solution; thus the soil-water suspension is being neutralized with the addition of this material. The soil-water mixture with sodium oxalate is then mixed in the malt mixer for 15 min, the material transferred to the sedimentation cylinder, and the test run as outlined herein.

The hydrometer is usually a type 152H (ASTM designation) and is calibrated to read grams of soil of a value of $G_s = 2.65$ in 1000 cm³ of suspension as long as no more than 60 g of soil is involved. The reading is, of course, directly related to the specific gravity of the solution. This particular hydrometer calibration is a considerable aid in the computations, as will be seen later. For this reason, this type of hydrometer is widely used, although other hydrometers can be used which read in

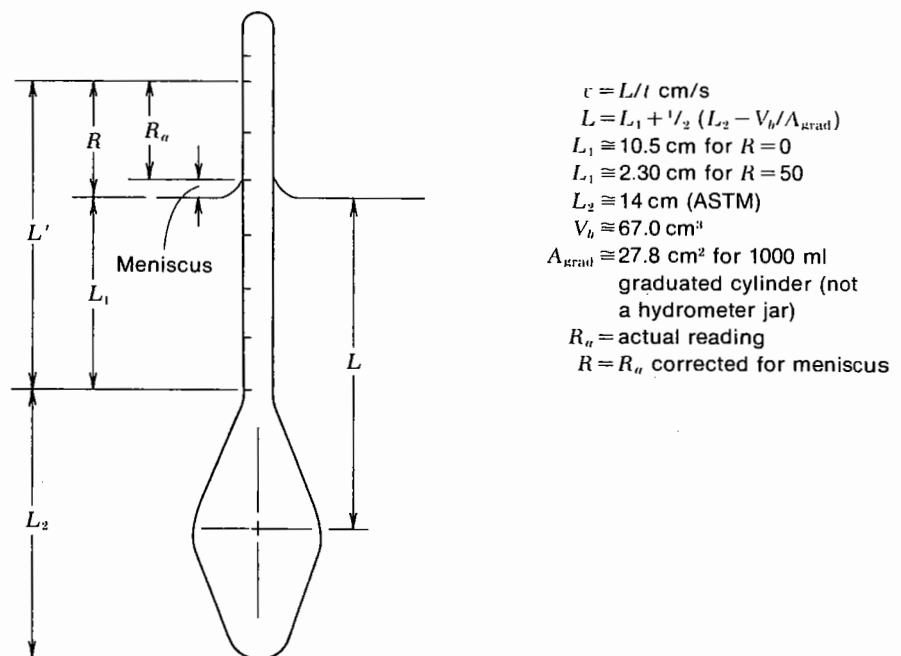


Figure 6-2
Hydrometer dimensions and terms.

Experiment Six

terms of the specific gravity of the soil-water suspension. For these latter hydrometers, care should be taken not to use much more than 60 g of soil in 1000 cm³ of water to avoid soil-grain interference during precipitation, with poor results obtained from application of Stokes' law.

The hydrometer displays the specific gravity of the soil-water suspension at the center of the bulb (refer to Fig. 6-2). Any soil grains larger than those still in suspension in the zone shown as L (the distance between the center of the volume of the bulb and the water surface) have fallen below the center of volume, and this constantly decreases the specific gravity of the suspension at the center of volume of the hydrometer. Also, it is obvious that since the hydrometer is a constant weight, the less the specific gravity of the suspension, the deeper the hydrometer will sink into the suspension (the larger the distance L will become). Remember, too, that for water the specific gravity (or density) decreases as the temperature rises (or falls) from 4°C. This also will cause the hydrometer to sink deeper into the suspension.

Since L represents the distance particles fall in some time interval t , and recalling that the velocity term in Eq. (6-2) is distance divided by time, it is evident that the velocity of fall of the particles is

$$v = \frac{L}{t}$$

Thus it is necessary to find L corresponding to some elapsed time t so that the velocity for use in Stokes' equation can be found.

To find L , measure the distance L_2 and several values of the variable distance L_1 (refer to Fig. 6-2) using a scale. Next, using a graduated sedimentation cylinder of known cross-sectional area A , submerge the hydrometer bulb and determine the change in cylinder reading. This will be the volume of the hydrometer bulb V_b . Now compute L in centimeters if L_1 and L_2 are in centimeters and V_b is in cubic centimeters

$$L = L_1 + \frac{1}{2} \left(L_2 - \frac{V_b}{A} \right) \quad (6-3)$$

The $-V_b/A$ term of Eq. (6-3) takes into consideration that the soil-water suspension rises the amount V_b/A when the hydrometer is placed in the sedimentation cylinder. Thus the center of volume is displaced upward the amount $\frac{1}{2}(V_b/A)$ of Eq. (6-3). By plotting a curve of hydrometer readings (which is related to the corresponding values of L_1) versus L , one may obtain L for any hydrometer reading R . Actually, since the curve is essentially linear, only about three points are necessary to establish the curve of R versus L . Since the 152H hydrometer is of reasonably standard dimensions, the L values can be obtained once for all with a precision adequate for most soil mechanics work, as in Table 6-5.

The hydrometer reading must not be corrected for use in Eq. (6-3) *except for meniscus* (in a turbid suspension, one must read to the top of the meniscus). The reason for using only this correction is that the velocity of fall is of interest and the actual reading is related to the distance L that the particles have fallen regardless of temperature, specific gravity of solution, or any other variable.

If one knows the particle diameter and percentage of soil remaining in suspension—which is, in this case, also the percent finer—sufficient data are available to plot a grain-size distribution curve.

The percent finer is related directly to the hydrometer reading of the 152H hydrometer since it reads the grams of soil still in suspension directly if the specific weight of the soil grains is 2.65 gm/cm³ and the water is 1.00 gm/cm³. The dispers-

ing agent will have some effect on the water and, additionally, the temperature of the test will be around 20°C and the G_s of the soil grains is not likely to be 2.65; thus, corrections will be required to the actual hydrometer reading to obtain the correct "reading" of the grams of soil still in suspension at any given instant of elapsed time.

The temperature can be kept at a single-value variable by using a water bath (if available), but this is a convenience not a necessity. The effect of water impurities and the dispersal agent on hydrometer readings can be obtained by using a sedimentation cylinder of water from the same source and with the same quantity of dispersing agent as used in the soil-water suspension to obtain a "zero correction." This jar of water should be at the same temperature as that of the soil-water suspension. A reading of less than zero in the standard jar of water is recorded as a (-) value; a reading between 0 and 60 is recorded as a (+) value. *All readings are taken to the top of the meniscus* in both standard jar (clear water) and soil suspension (turbid water).

If the temperature is too high in both the soil and "standard" cylinders, the density of the water will be equally less and the hydrometer will sink too deep. If both jars are at the same temperature, the effect will be the same; thus, a single-value temperature correction can be used for the soil-water suspension, which can be obtained from Table 6-3. Since the zero correction is (\pm) and the temperature correction is also (\pm) and the sign is as in Table 6-3, the corrected hydrometer reading for grams of soil in suspension is computed as

$$R_c = R_{\text{actual}} - \text{zero correction} + C_T \quad (6-4)$$

Now that a corrected hydrometer reading has been obtained (but one that does not include the fact that the specific gravity of the soil solids may not be 2.65), the percent finer can be computed by simple proportion (if $G_s = 2.65$) as

$$\text{Percent finer} = \frac{R_c}{W_s} 100 \quad \text{percent}$$

where R_c = grams of soil in suspension at some elapsed time t [it is also the corrected hydrometer reading from Eq. (6-4)]

W_s = weight of original soil sample placed in suspension, g

If G_s is not equal to 2.65, one can compute a multiplier a to use with Eq. (6-5) by proportion as follows:

$$\frac{a}{G_s/(G_s - 1)} = \frac{1}{2.65/(2.65 - 1)}$$

Solving for a , one obtains

$$a = \frac{G_s(1.65)}{(G_s - 1)2.65} \quad (6-6)$$

The percent finer, when $G_s \neq 2.65$, is computed as

$$\text{Percent finer} = \frac{R_c a}{W_s} 100 \quad \text{percent} \quad (6-7)$$

Typical values of a are given in Table 6-2.

For computational purposes, Eq. (6-2) is usually rewritten using L in cm and t in min to obtain D in mm as follows:

$$D = \sqrt{\frac{30\eta}{980(G_s - G_w)}} \frac{L}{t} \quad (6-8)$$

which can be simplified even further to

$$D = K \sqrt{\frac{L}{t}} \quad \text{mm} \quad (6-8a)$$

Since all but L/t are independent of the problem *except for the temperature* of the suspension, one can evaluate $K = f(T, G_s, \eta)$ once for all, as in Table 6-4.

Procedure *This Should Be a Group Project*

METHOD A: Use the following procedure where 80 to 90 percent of the material passes the No. 200 sieve. Where a lesser amount passes the No. 200 and/or (+) No. 10 material is present, consider using Method B, following.

1. Take exactly 50 g of oven-dry, well-pulverized soil (as used in the sieve analysis, if specified) and mix with 125 ml of 4% NaPO_3 solution.¹ A 4% sodium metaphosphate solution can be made by mixing 40 g of dry material with enough water to make 1000 ml. The solution should be freshly mixed and in any case not over 1 month old.
2. Allow the soil mixture to stand about 1 h (ASTM suggests 16 h for clayey soils, but this is generally not necessary). Transfer the mixture to a malt-mixer cup and add tap water² until the cup is two-thirds full. Mix for 1 min if soaked for 10 to 16 h, otherwise mix for 3 to 5 min.
3. Transfer all the contents of the mixer cup to the sedimentation cylinder, being very careful not to lose any material. Add tap water to fill the cylinder to the 1000-ml mark. Prepare the control jar (Fig. 6-1) with tap water and 125 ml of the 4% solution (or same as used in step 1). Be sure the tap water temperature is adjusted so that both the sedimentation and control cylinders are at the same temperature.
4. Take a No. 12 rubber stopper (use the palm of your hand if stopper is not available) and cap the cylinder of soil suspension and carefully agitate for about 1 min. Set the jar down, remove the stopper, immediately insert the hydrometer and take readings at elapsed times of
1, 2, 3, and 4 min³

Also take a thermometer reading

Place the hydrometer and thermometer in the control jar (which should be within 1°C of soil-water suspension). Take a meniscus reading in the control jar on the hydrometer.

It is usual to leave the hydrometer in the soil-water suspension for the first two readings, and then remove and reinsert for each of the other two readings.

¹If sodium silicate (water glass) is used as a dispersing agent, use between $\frac{1}{2}$ and 1 ml of concentrated solution with 125 ml of water.

²ASTM requires distilled or demineralized water, but tap water should be satisfactory since both containers are the same. The dispersing agent will "pollute" any water used.

³Note that if you use the ASTM procedure, you may not be able to take some of the first readings. ASTM uses 100 to 120 g of soil in suspension which the hydrometer cannot read until enough has settled so that only 50 to 60 g is in suspension

However, little error is introduced if it is left in the suspension for all four readings. When placing the hydrometer in the suspension, avoid unnecessary agitation by proceeding slowly, so that about 10 s is required for the operation.

5. Replace the No. 12 stopper and reagitrate the suspension and take another series of 1-, 2-, 3-, and 4-min readings. Repeat as necessary until you obtain two sets of readings which agree within 1 unit of each other for all four readings. If you do not use a jar bath, continue the test by taking readings as in step 6.

If you use a jar bath, once more agitate the suspension, and place it in the jar bath along with the control jar.

6. Take a 4-min reading in the jar bath and compare to the earlier 4-min readings; if sufficient agreement is made continue, otherwise, reagitrate and redo until satisfactory agreement is reached. When agreement between readings is reached (within 1 unit), take additional readings at elapsed times of:

8, 15, 30, 60 min, and 2, 4, 8, 16, 32, 64, 96 h

Times beyond the first two hours are approximate and any elapsed times which give a good spread of plotted points will be satisfactory.

Record the temperature of the soil-water suspension to the nearest 1°C for each hydrometer reading.

The test may be terminated prior to 96 h at the discretion of the instructor. If no special termination instructions are given, the test should be continued until (make computations) the grain size D still in suspension is on the order of 0.001 mm.

Between hydrometer readings, store the hydrometer and thermometer in the control jar (which should be at the same temperature).

7. Go to "Computations."

METHOD B: This method should be used to obtain both the mechanical and hydrometer analysis for soils containing appreciable coarser material. It may be used for all soils but is longer than Method A above, and the procedure of Experiment No. 5. For this reason Method A and/or Experiment No. 5 is to be preferred for student laboratories.

1. Oven-dry an appropriate amount of soil based on the maximum-size aggregate. This quantity will be about 300 to 1000 g *larger* than the following quantities:

500 g for maximum-size aggregate 10 mm and smaller (actual sample weight)

5000 g for maximum-size aggregate 75 mm and larger

Use linear interpolation to next larger 1000 g for intermediate sizes

Call the actual sample weight W_o —and use either 500, 1000, 2000 g, etc. Be sure the total sample oven-dried is larger; e.g., for a 500-g sample, oven-dry a sample of 700 to 1000 g. The 500 g is W_o .

2. Carefully wash W_o through the No. 200 sieve as in Experiment No. 5 and oven-dry the residue. Perform a mechanical sieve analysis using 5 to 8 sieves with a size range from the maximum particle size and including the No. 50 and/or No. 100 and the No. 200 sieves. Whether the No. 50 or No. 100 (or both) are used will depend on the stack size and the estimated grain sizes in the material.

Plot the grain-size curve using steps 1 through 9 of "Procedure" in Experiment No. 5. Lightly pencil in the distribution curve at this point.

3. From the grain size distribution curve just drawn, obtain the percent finer than the No. 50 or the No. 100 sieve. Use the No. 100 sieve if both sieves are used. The No. 200 sieve is included to check the efficiency of washing the sample. Call the percent finer value obtained N_i .

4. Obtain a smaller, representative, oven-dry sample from the oven-dry batch of an exact weight W_i , obtained as

$$W_i = \frac{50}{N_i} \quad (\text{g})$$

where N_i = decimal percent finer from step 3. This will produce a hydrometer sample between 50 and 60 g in the reading range of the 152H hydrometer and with all the particle sizes in the range of validity of Stokes' law.

5. Pulverize this sample as best you can and dry sieve through the percent finer sieve (50 or 100) into a pan. Save the dry material in the pan. Recover the residue (+) material and mix with water in a porcelain dish to soak the soil thoroughly. Let this material soak for several hours (up to 24 h) so that any lumps are thoroughly softened. Next wash the soaked material through the same percent finer sieve into the sieve pan with the earlier dry material. Use as little water as possible, but be sure to wash the sample clean. Save all the water [and (-) soil] in a beaker or other container, depending on the quantity, and let stand.
6. Recover the washed residue (+) material, oven-dry and weigh to obtain the weight W_f . Use the initial dry weight W_i of step 4 and repeat step 2 for a second grain-size distribution curve data set. Lightly plot this data also on the curve of step 2. Note that only a happy coincidence would produce identical curves. Compute W_s as

$$W_s = W_i - W_f$$

7. If the washed (-) material occupies more than 1000 ml, let the soil-water suspension stand and then decant the clear top water until a volume of 400 to 600 ml remains.
8. Transfer the soil-water suspension to a malt-mixer cup and mix for 5 to 10 min. Pour the material into the 1000-ml sedimentation jar. Add 125 ml of dispersing agent and enough additional tap water to fill the cylinder to the 1000-ml mark. Be very careful not to lose any soil in any of the transferring operations and see that the mixer cup is washed clean into the sedimentation jar.
9. Refer to Method A, steps 4 and following, for the remainder of the hydrometer test.

Computations

(BOTH METHOD A AND METHOD B)

1. Apply the meniscus correction to the hydrometer readings and enter Table 6-5 to obtain values of L . If G_s is not known (or furnished by instructor), assume a reasonable value between 2.68 and 2.74. Select a value to avoid unnecessary interpolation in any assumption. With G_s and the test temperature for any hydrometer reading, enter Table 6-4 to obtain the K value. With the values of K , L , and the elapsed time t , for those readings, compute the values of D using Eq. (6-8a). Use Fig. 6-3 and "Sample Computations" as a guide.
2. Using a corrected value of R_c from Eq. (6-4) in Eq. (6-7), compute the percent finer for the corresponding particle diameters D of step 1 above.
3. Use the data from steps 1 and 2 above and plot the percent finer vs. grain size—either on the mechanical sieve-analysis curve from Experiment No. 5 (or from Method B) or on a new curve data sheet, depending on the soil used.

Carefully note that percent finer is independent of quantity of soil used so that results from a mechanical analysis can be combined with the hydrometer analysis. If this is done, keep in mind that the mechanical analysis uses a larger sample and should be given more weight when drawing the distribution curve across the discontinuous region between the two curve branches. If Method B is used refer to step 3b following.

Be sure to identify the plotted points as from the mechanical or hydrometer analysis somewhat similar to Fig. 5-4.

THE FOLLOWING COMPUTATIONS SHOULD BE MADE ONLY IF METHOD B IS USED:

3. (a) Compare the percent passing the sieve used for obtaining the soil for the hydrometer test with the percent passing in the W_o sample. Do you have any opinion for your report for differences less than $\pm 2.5\%$; $\pm 5\%$; any larger percentage? (b) Adjust the mechanical analysis part of the distribution curve for the two sets of mechanical analysis data using a *weighted* average based on sample weights as

$$\text{Weighted \% finer } D_i = \frac{W_o(\% \text{ finer } D_{i(W_o)}) + W_t(\% \text{ finer } D_{i(W_t)})}{W_o + W_t}$$

Plot these adjusted percent finer values (which should fall between the two percent finer values for that grain size D_i) and draw the final mechanical analysis portion of the laboratory curve using a heavy line.

4. Show on the curve the D_{85} , D_{60} , D_{30} , and D_{10} sizes, if possible. For the hydrometer branch of the curve show the percent clay.
5. In your report:
 - a. Derive Eq. (6-8) from Eq. (6-2); also verify Eq. (6-4).
 - b. Comment on the effect of using 60 g or more of soil and/or with $G_s > 2.65$ instead of the quantity (approximately 50 g) of soil used.
 - c. Why is there no meniscus reading correction in Eq. (6-4)?
 - d. What is the effect of using an erroneous G_s of, say, 2.68 when the correct value is 2.70?
 - e. Why do C_c and C_u have no meaning when more than 12 percent passes the No. 200 sieve?
 - f. State two or more uses for hydrometer test data.

Sample Computations

From line 1 of Fig. 6-3 (typical data sheet)

$$R_c = R_a - \text{zero correction} + C_T \text{ (} C_T \text{ from Table 6-3 at } 22^\circ\text{C)}$$

$$R_c = 49 - 3 + 0.40 = 46.4 \quad a = 0.99 \text{ (from data sheet)}$$

$$\text{Percent finer} = \frac{R_c a}{W_s} \times 100 = \frac{46.4(0.99)(100)}{50.0} = 91.9\%$$

$$\text{Hydrometer corrected for meniscus only} = 49 + 1 = 50$$

$$L = 8.1 \text{ (from Table 6-5 at } R = 50)$$

$$v = \frac{L}{t} = \frac{8.1}{1} = 8.1$$

$$K = 0.0131 \text{ (from Table 6-4 using } T \text{ and } G_s)$$

$$D = K\sqrt{L/t} = 0.0131\sqrt{8.1} = 0.037 \text{ mm} \quad \text{(Do not compute more digits than you can plot on the graph using log scale)}$$

From line 3 of data sheet

$$R_c = 43 - 3 + 0.4 = 40.4$$

$$\text{Percent finer} = \frac{40.4(0.99)(100)}{50.0} = 80.0\%$$

$$\text{Hydrometer corrected for meniscus} = 43 + 1 = 44$$

$$L = 9.1 \text{ (from Table 6-5)}$$

$$K = 0.0131 \text{ (from Table 6-4)}$$

$$D = 0.0131\sqrt{9.1/3} = 0.023 \text{ mm}$$

From last line of data sheet

$$R_c = 10 - 3 + 1.0 = 8.0$$

$$\text{Percent finer} = \frac{8.0(0.99)(100)}{50.0} = 15.8\%$$

$$\text{Hydrometer corrected for meniscus} = 10 + 1 = 11$$

$$L = 14.5 \text{ (from Table 6-5)}$$

$$K = 0.0127 \text{ (from Table 6-4)}$$

$$D = 0.0127\sqrt{14.5/2850} = 0.0009 \text{ mm}$$

GRAIN SIZE ANALYSIS-HYDROMETER METHOD

Data Sheet 7

Project Hydrometer Analysis Job No. ~
 Location of Project Bradley University Boring No. ~ Sample No. ~
 Description of Soil Brown Silty Clay Depth of Sample ~
 Tested By JEB Date of Testing 3/4/76

Hydrometer analysis

Hydrometer no. 152H G_s of solids = 2.72 $u =$ 0.99
 Dispersing agent NaPO₃ (Calgon) Amount 4% in 125 ml Wt. of soil, W_s 50.0 g
 Zero correction +3.0 Meniscus correction 1.0

Date	Time of reading	Elapsed time, min	Temp., °C	Actual Hyd. reading R_a	Corr. Hyd. reading R_c	% Finer	Hyd. Corr. only for meniscus, R	L from Table 6-5	$\frac{L}{I}$	K from Table 6-4	D , mm
3-4	P.m.	1	22°	49	46.4	91.9	50	8.1	8.1	0.0131	0.037
		2		47	44.4	87.9	48	8.4	4.2		0.027
		3		43	40.4	80.0	44	9.1	3.03		0.023
		4		42	39.4	78.0	43	9.2	2.3		0.020
		8		37	34.4	68.1	38	10.1	1.26		0.015
		16		31	28.4	56.2	32	11.1	0.69		0.011
		30		26	23.4	46.3	27	11.9	0.397		0.0082
	4:30	60		24	21.4	42.4	25	12.2	0.203		0.0059
	5:35	125		21	18.4	36.4	22	12.7	0.102	0.0131	0.0042
	9:00	330	23°	18	15.7	31.1	19	13.2	0.040	0.0129	0.0026
3-5	A.m.										
	8:00	990	23°	16	13.7	27.1	17	13.5	0.014	0.0129	0.0015
	P.m.										
	3:00	1410	24°	14	12.0	23.8	15	13.8	0.0098	0.0127	0.0013
3-6	P.m.										
	3:00	2850	24°	10	8.0	15.8	11	14.5	0.0051	0.0127	0.0009
Line 2: % Finer = $\frac{44.4(0.99)(100)}{50} = 87.9\%$											
$D = 0.0131 \sqrt{\frac{8.4}{2}} = 0.027 \text{ mm}$											
** Compute D to 2 significant digits for plotting											

$$R_c = R_{\text{actual}} - \text{zero correction} + C_T$$

$$\% \text{ finer} = R_c(u)/W_s$$

$$D = K\sqrt{L/t}$$

Figure 6-3

Typical data from hydrometer test. Percent finer vs. D is plotted on Fig. 5-4, curve A.

Table 6-1 Properties of Distilled Water

Temp (°C)	Unit weight of water (g/cm ³)	Viscosity of water (poises) ¹
4	1.00000	0.01567
16	0.99897	0.01111
17	0.99880	0.01083
18	0.99862	0.01056
19	0.99844	0.01030
20	0.99823	0.01005
21	0.99802	0.00981
22	0.99780	0.00958
23	0.99757	0.00936
24	0.99733	0.00914
25	0.99708	0.00894
26	0.99682	0.00874
27	0.99655	0.00855
28	0.99627	0.00836
29	0.99598	0.00818
30	0.99568	0.00801

$$^1\text{Poise} = \frac{\text{dyne} \cdot \text{s}}{\text{cm}^2} = \frac{\text{g}}{\text{cm} \cdot \text{s}}$$

Table 6-2 Correction Factors *a* for Unit Weight of Solids

Unit weight of soil solids (g/cm ³)	Correction factor <i>a</i>
2.85	0.96
2.80	0.97
2.75	0.98
2.70	0.99
2.65	1.00
2.60	1.01
2.55	1.02
2.50	1.04

Table 6-3 Temperature Correction Factors *C_T*

Temp. (°C)	<i>C_T</i>
15	-1.10
16	-0.90
17	-0.70
18	-0.50
19	-0.30
20	0.00
21	+0.20
22	+0.40
23	+0.70
24	+1.00
25	+1.30
26	+1.65
27	+2.00
28	+2.50
29	+3.05
30	+3.80

Table 6-4 Values of K for Use in Eq. (6-8a) for Several Unit Weights of Soil Solids and Temperature Combinations

Temp. (°C)	UNIT WEIGHT OF SOIL SOLIDS (g/cm ³)							
	2.50	2.55	2.60	2.65	2.70	2.75	2.80	2.85
16	0.0151	0.0148	0.0146	0.0144	0.0141	0.0139	0.0137	0.0136
17	0.0149	0.0146	0.0144	0.0142	0.0140	0.0138	0.0136	0.0134
18	0.0148	0.0144	0.0142	0.0140	0.0138	0.0136	0.0134	0.0132
19	0.0145	0.0143	0.0140	0.0138	0.0136	0.0134	0.0132	0.0131
20	0.0143	0.0141	0.0139	0.0137	0.0134	0.0133	0.0131	0.0129
21	0.0141	0.0139	0.0137	0.0135	0.0133	0.0131	0.0129	0.0127
22	0.0140	0.0137	0.0135	0.0133	0.0131	0.0129	0.0128	0.0126
23	0.0138	0.0136	0.0134	0.0132	0.0130	0.0128	0.0126	0.0124
24	0.0137	0.0134	0.0132	0.0130	0.0128	0.0126	0.0125	0.0123
25	0.0135	0.0133	0.0131	0.0129	0.0127	0.0125	0.0123	0.0122
26	0.0133	0.0131	0.0129	0.0127	0.0125	0.0124	0.0122	0.0120
27	0.0132	0.0130	0.0128	0.0126	0.0124	0.0122	0.0120	0.0119
28	0.0130	0.0128	0.0126	0.0124	0.0123	0.0121	0.0119	0.0117
29	0.0129	0.0127	0.0125	0.0123	0.0121	0.0120	0.0118	0.0116
30	0.0128	0.0126	0.0124	0.0122	0.0120	0.0118	0.0117	0.0115

Table 6-5 Values of L (Effective Depth) for Use in Stokes' Formula for Diameters of Particles for ASTM Soil Hydrometer 152H

Original hydrometer reading (corrected for meniscus only)	Effective depth L (cm)	Original hydrometer reading (corrected for meniscus only)	Effective depth L (cm)	Original hydrometer reading (corrected for meniscus only)	Effective depth L (cm)
0	16.3	21	12.9	42	9.4
1	16.1	22	12.7	43	9.2
2	16.0	23	12.5	44	9.1
3	15.8	24	12.4	45	8.9
4	15.6	25	12.2	46	8.8
5	15.5	26	12.0	47	8.6
6	15.3	27	11.9	48	8.4
7	15.2	28	11.7	49	8.3
8	15.0	29	11.5	50	8.1
9	14.8	30	11.4	51	7.9
10	14.7	31	11.2	52	7.8
11	14.5	32	11.1	53	7.6
12	14.3	33	10.9	54	7.4
13	14.2	34	10.7	55	7.3
14	14.0	35	10.5	56	7.1
15	13.8	36	10.4	57	7.0
16	13.7	37	10.2	58	6.8
17	13.5	38	10.1	59	6.6
18	13.3	39	9.9	60	6.5
19	13.2	40	9.7		
20	13.0	41	9.6		

Experiment No. 7

SPECIFIC GRAVITY OF SOIL SOLIDS

References

AASHTO T100-70
ASTM D854-58

Objective

To familiarize the student with a general method of obtaining the specific gravity of a mass of any type of material composed of small particles which has a specific gravity greater than 1.00. The project is specifically applicable to soil and fine aggregates (or sand) as used in concrete and asphalt mixtures.

Equipment

Volumetric flask, preferably 250 or 500 ml (see Fig. 7-1)
Vacuum pump or aspirators for supplying a vacuum
Mortar and pestle
Balance weighing to 0.1 g
Supply of deaired, temperature stabilized water ¹
Optional: Container of ice water and mechanical (malt) mixer

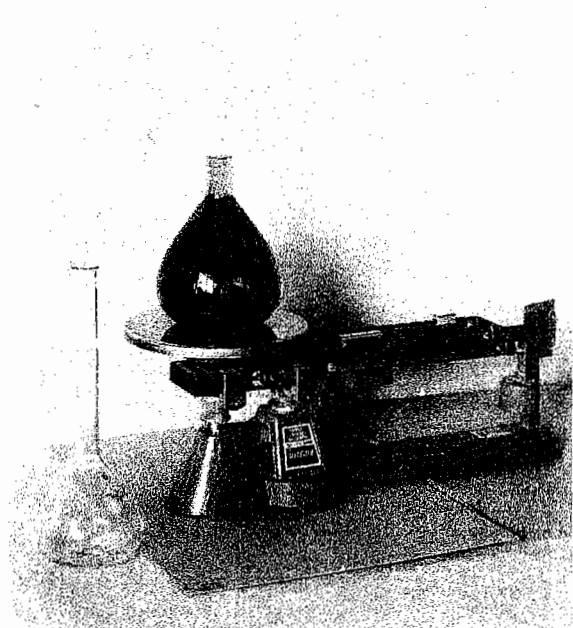


Figure 7-1

Balances and volumetric flask (500-ml shown) for the specific-gravity test. Also shown is the weighing of the value of W_{bus} after careful deairing of the soil-water suspension.

¹Prior to class a supply of tap (or distilled) water should be collected and deaired (about 1000 ml/group); use tap water, hot water, and/or ice water to affect a stabilized water temperature. Also check and oven-dry the volumetric flasks if necessary.

General Discussion

The specific gravity G_s of a soil without any qualification¹ is taken to be the average value for the *soil grains*. If numerical values are given in a discussion where it may not be clear to what the specific gravity is referred, the magnitude of the values may indicate the correct usage since the specific gravity of the soil grains will always be larger than the bulk specific gravity based on inclusion of the soil voids in the computation [either full of air (dry) or full of water or partly full of water].

A value of specific gravity is necessary to compute the void ratio of a soil, it is used in the hydrometer analysis, and it is useful to predict the unit weight of a soil. Occasionally, the specific gravity may be useful in soil mineral classification; e.g., iron minerals have a larger value of specific gravity than silicas.

The specific gravity of any substance is defined as the unit weight of the material divided by the unit weight of distilled water at 4°C. Thus, if one is considering only the soil grains, one obtains G_s as

$$G_s = \frac{\gamma_{\text{material}}}{\gamma_{\text{water at 4°C}}} \quad (7-1)$$

The same form of equation is used if bulk specific gravity is sought, the only difference being in the value of γ_{material} . The specific gravity of a material can also be computed using any ratio of weight of substance to the weight of water as long as equal volumes are involved:

$$G_s = \frac{W_s/V}{W_w/V} \quad (7-2)$$

From Eq. (7-2) it is evident that this is so since the volume term cancels. Note, however, that if one does not cancel V in Eq. (7-2), one obtains Eq. (7-1).

The problem consists in obtaining the volume of a known weight of soil grains and dividing this by the weight of the same volume of water, i.e., applying Eq. (7-2) since this form is easier to visualize as well as to obtain values for in a laboratory. The volume of a known weight of soil grains can be obtained by using a container of known volume and the Archimedes principle that a body submerged in a volume of water will displace a volume of water equal to the volume of the submerged body.

The container of known volume is the volumetric flask, which holds a standard volume of distilled water at 20°C. At temperatures above this value, the volume will be slightly more; below 20°C, the volume will be slightly less. Since the volume change is small for small temperature deviations and it is relatively easy to hold the test temperature close to 20°C, an approximate temperature correction for small temperature deviations can be applied in the required computations which produces satisfactory accuracy without recourse to experimentally determining the change in flask volume with temperature. Alternatively, one may develop a calibration curve for any given volumetric flask by:

1. Carefully cleaning
2. Fill with distilled, demineralized, or tap water at known temperatures
3. Make a plot of weight (W_{bw}) vs. $T^\circ\text{C}$ (use about 4 points at, say, 16, 20, 24, and 28°C)

¹Some texts use the symbol G_s to indicate the specific gravity of the soil grains; others use the symbol G . The author prefers to use G_s .

Tap water instead of distilled water is often used in this test (at least for routine work); again relatively little error is involved. The error introduced from using tap water can be determined by filling the volumetric flask with tap water to the volume mark and obtaining the temperature and weight. From this data and the weight of the empty flask, one can compute the density of the tap water and compare with the density of distilled water in tables such as Table 6-1. Note that if the temperature is not exactly 20°C the volume of the flask will require calibration as previously outlined. Generally, if the density error is less than 0.001, it can be neglected.

Since the laboratory work to determine the specific gravity of the soil using the volumetric flask is a somewhat indirect method (that is, one indirectly displaces the volume of the material), we shall derive the expression to compute the specific gravity:

1. Let W_b = weight of the empty (and dry) volumetric flask.
2. Let W_{bw} = weight of bottle + distilled or tap water to the volume mark on the flask.
3. Now place the weight of soil W_s into the flask and again fill the flask to the volume mark and weigh. Let this value be W_{bws} .
4. Recalling that one is dealing with a constant volume, if no water was displaced from the bottle when W_s was added, the weight would be

$$W_T = W_{bw} + W_s$$

Since water is displaced (i.e., an amount to equal W_{bw} cannot be added to the bottle since W_s occupies part of the volume) to bring the volume to the volume mark, the weight of water which cannot be poured into the bottle must have been

$$W_w = W_T - W_{bws} = W_{bw} + W_s - W_{bws}$$

5. From the definition of G_s in Eq. (7-2), and if one does not have to consider any density (and volume) changes with temperature, the specific gravity is

$$G_s = \frac{W_s}{W_w}$$

since equal volumes are involved.

We may also write

$$G_s = \frac{W_s}{W_{bw} + W_s - W_{bws}}$$

A slight increase in precision to account for temperature effects on the density of water can be obtained by rewriting Eq. (7-3) as

$$G_s = \frac{\alpha W_s}{W_{bw} + W_s - W_{bws}} \quad (7-3)$$

where α , the temperature correction coefficient, is computed as

$$\alpha = \frac{\gamma_T}{\gamma_{20^\circ C}} \quad (7-4)$$

and is the ratio of the unit weights of water at the temperature T of the test and

at 20°C formed such that the value of G_s obtained at temperature T (which will be too large if $T > 20^\circ\text{C}$) is appropriately reduced. Note that α is also G_w of the water at the test temperature T .

Typical values of the correction factor, α , are as follows:

$T, ^\circ\text{C}$	α	$\gamma_w, \text{g/cm}^3$
16	1.0007	0.99897
18	1.0004	0.99862
20	1.0000	0.99823
22	0.9996	0.99780
24	0.9991	0.99732
26	0.9986	0.99681

The most serious source of error in this experimental determination of the specific gravity is not temperature (especially if the test temperature is kept between 18 and 22°C) or whether distilled or tap water is used. The major source of error will occur from not properly deairing the soil-water mixture. Water contains, under normal circumstances, dissolved air. The soil particles will also contain air, and if the air is not removed from both of these materials, the volume of air will result in a decrease in the weight W_{bws} . This in turn will result in too small a computed value of G_s , that is, $W_{bw} + W_s - W_{bws}$ will be too large. Another source of error, which can be important, is the use of weights obtained from poorly adjusted balances or weighings obtained from two different balances.

Deairing the soil-water mixture is accomplished by applying a vacuum and/or heating. Vacuum is usually sufficient for sands, silts, and clays. For organic soils it will be necessary to boil the soil-water mixture about 30 min, adding water as necessary to keep the flask about half full.

The length of time the vacuum should be applied may range from a few minutes to

- 6 to 8 h for very plastic soils
- 4 to 6 h for soils of low plasticity

The efficiency of air removal may be improved for any soil by boiling for about 10 min with care taken not to boil the sample dry or lose any soil material. Deairing can be checked by

- (1) Applying vacuum to a half-to-three-quarters-filled flask of soil-water mixture for a period of time, then,
- (2) Filling flask to about 20 mm of volume mark with temperature-stabilized *deaired* water.
- (3) Reapplying the vacuum for several minutes and with a colored grease pencil marking the water level in the neck, then,
- (4) Carefully pull the stopper to break the vacuum and, if the water level drops not more than 3 mm, the deairing should be sufficient.

Typical values of G_s which can be used as a guide in determining whether the test results are correct, are as follows:

Type of soil	G_s
Sand	2.65-2.67
Silty sand	2.67-2.70
Inorganic clay	2.70-2.80
Soils with micas or iron	2.75-3.00
Organic soils	Variable but may be under 2.00

Procedure

1. Mix 100 to 120 g (exact weight is not important at this point) of air-dry soil with water in an evaporating dish to form a creamy paste. If a malt mixer is not used, soak soil 20 to 30 min (ASTM suggests 12 h for oven-dried samples).
2. Optional: Transfer the paste to malt-mixer container and add water to make about 200 ml of soil-water mixture. Mix the soil-water mixture for about 5 to 10 min. If you do this step, you will need a 500-ml volumetric flask.
3. Weigh the dry volumetric flask, then carefully fill to the volume mark with deaired water. Use care not to reintroduce air into the water by splashing. Alternatively, apply vacuum for a few minutes after filling the flask about three-quarters full. With the water level at the volume mark (not meniscus) and the neck inside above the volume mark dry, weigh the flask and record the weight as W_{bw} . Take a temperature reading so the soil-water mixture will be at the same temperature \pm not more than 1°C. Do this while waiting for the soil to soak or while your soil is being mixed in the mechanical mixer.

Omit this step if a calibration curve for the flask is available.

4. After 15 to 30 min, transfer the soaked soil from the evaporating dish to the volumetric flask. Wash all the soil into the flask. Add sufficient temperature-stabilized water to fill the flask two-thirds to three-quarters full. Do not over-fill as the vacuum efficiency will be markedly reduced.
5. Attach the flask to a high vacuum for at least 10 min. During this time gently agitate the mixture by carefully shaking and turning the bottle. Observe that the reduced air pressure in the flask causes the water to "boil" (see Fig. 7-2). Check the vacuum efficiency as outlined in the "General Discussion." If the air has not been sufficiently removed, let the flask stand for a few minutes so that the top water clears, then pour out enough water so that a vacuum can be efficiently applied. This step can take several hours, however; for student laboratories, check with the instructor after a reasonable amount of time for next step.

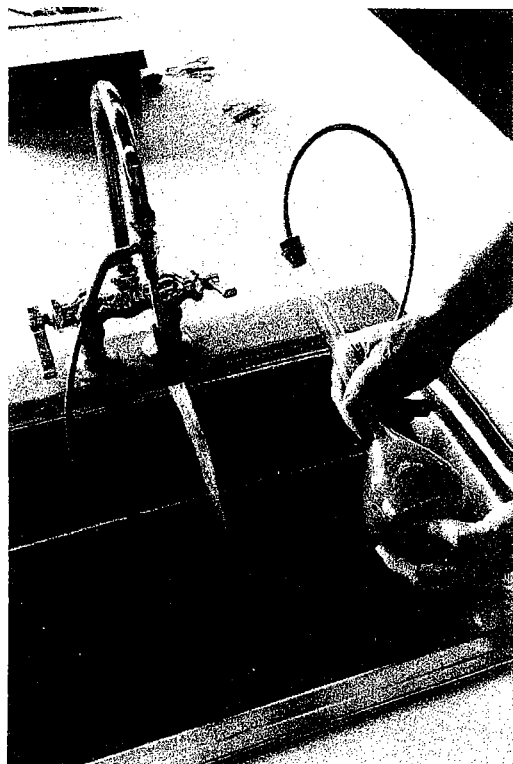


Figure 7-2

Using an aspirator connected to sink faucet to supply the deairing vacuum. Vacuum intensity is determined by faucet flow. Note flask is about two-thirds full and contents are "boiling" (bubbles and foaming inside flask). When using this device *be very careful to unstopper the flask prior to shutting faucet* so that water in the system is not drawn into the flask due to large pressure differential existing between aspirator and exit piping.

SPECIFIC GRAVITY OF SOIL SOLIDS (G_s)

Data Sheet 8

Project EXP. No. 7 Job No. ~
 Location of Project Bradley University Boring No. ~ Sample No. ~
 Description of Soil Brown Silty Clay Depth of Sample ~
 Tested By JEB Date of Testing 12-3-75

Test no.	1	2		
Vol. of flask at 20°C	500 ml.	500 ml.		
Method of air removal ^a	Vacuum	Aspir.		
Wt. flask + water + soil = W_{bws}	753.66	754.69		
Temperature, °C	23°	23°		
Wt. flask + water ^b = W_{bw}	693.27	693.27		
Evap. dish no.	5	4		
Wt. evap. dish + dry soil	350.11	368.49		
Wt. of evap. dish	254.52	270.52		
Wt. of dry soil = W_s	95.59	97.979		
$W_w = W_s + W_{bw} - W_{bws}$	35.20	36.55		
$G_s = \alpha W_s / W_w (0.99756)$	2.71	2.67		

^aIndicate vacuum or aspirator for air removal.

^b W_{bw} is the weight of the flask filled with water at same temp. $\pm 1^\circ\text{C}$ as for W_{bws} or value from calibration curve at T of W_{bws} .

Remarks $\gamma_o = \frac{2.71}{2.67} = 1.01 < 1.02$ o.k.

Average specific gravity of soil solids (G_s) = 2.69

Figure 7-3

Data from a specific gravity test (two trials).

6. When the deairing process is complete (or has been terminated), carefully add water until the bottom of the meniscus is exactly at the volume mark. Be very careful not to reintroduce air into the flask when completing the filling operation. Use water from the deaired, temperature-stabilized supply. Carefully dry the neck of the flask above the calibration mark with a rolled paper towel or some other means.
7. Weigh the bottle and its contents to the nearest 0.01 g (by estimation) to obtain W_{bws} . Be sure the temperature is within 1°C of the temperature used for obtaining W_{bw} (unless a calibration curve is used).
8. Empty the flask and its contents into a deep evaporating dish or other container and oven-dry. *Be very careful not to lose any soil at this point.* Weigh the oven-dry soil to obtain W_s .
9. Compute G_s using Eq. (7-4). Compare the density of water used with that of distilled water and comment in your report on the effect of using tap water.
10. Repeat the sequence outlined (steps 1-9) for additional values of G_s until you have two values which are within 2 percent of each other, defined as follows:

$$\frac{\text{Largest value of } G_s}{\text{Smallest value of } G_s} \leq 1.02$$

When you have obtained this pair of values of G_s , average them, round off to the nearest 0.01 and report the value as G_s for the soil.

Experiment No. 8

CLASSIFICATION OF SOILS

References

- AASHTO (1971 or later ed.) Specifications M145-66: The Classification of Soils and Soil Aggregate Mixtures for Highway Construction Purposes, "Highway Materials," vol 1.
- Casagrande, A., (1948), Classification and Identification of Soils, *Transactions, ASCE*, vol. 113, pp. 901-991.
- Highway Research Board (1945), Classification of Highway Subgrade Materials, *Proceedings*, vol. 25, pp. 376-392
- Wagner, A. A. (1957), The Use of the Unified Soil Classification System by the Bureau of Reclamation, *Proc. 4th Int. Conf. Soil Mech. Found. Eng., London*, vol. 1, pp. 125-134.

Objective

To introduce the student to two methods of soil classification. Secondary benefits will be derived from the additional practice obtained in performing the required soil tests to provide the classification data.

Equipment

As required

General Discussion

There are several methods of soil classification—the Unified Soil Classification system, the American Association of State Highway and Transportation Officials (AASHTO) system, the U.S. Department of Agriculture (USDA) system, the ASTM system, and the Federal Aviation Agency (FAA) system to name a few. At present the state departments of transportation (formerly highway departments) use the AASHTO system of soil classification. The foundation engineer usually makes a visual inspection of the soil the primary basis for classification, but his terminology, which will not be considered here, is very similar to that of the Unified Soil Classification system.

All the classification systems of primary interest to the geotechnical engineer use the Atterberg limits (at least the liquid and plastic limits) with a partial-to-complete grain-size analysis and the process of elimination to classify a soil.

In all the classification systems, it is absolutely essential that a soil description should accompany the symbol classification, because a particular classification group symbol is too broad and too general a classification for specific soils.

Because there is a general similarity among the various methods and the Unified and AASHTO soil classification systems are more widely used than any of the other systems, this laboratory project will be concerned only with these two methods.

A. AASHTO SOIL CLASSIFICATION

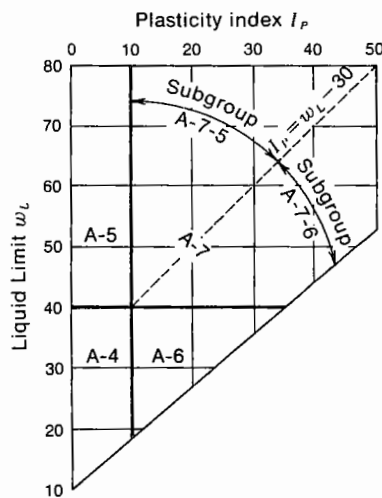
Figure 8-1a displays the AASHTO soil classification system in a convenient chart form. Figure 8-1b provides a rapid means of using the liquid and plastic limits to obtain the classification of the A-4 through A-7 soils where an essential classification factor is that more than 35 percent of the soil passes the No. 200 sieve. This chart can also be used to obtain the subgroup classification of the A-2 soils where the essential factor is that *not more than 35 percent passes the No. 200 sieve*. The

General classification	Granular materials (35 percent or less of total sample passing No. 200)							Silt-clay Materials (More than 35 percent of total sample passing No. 200)			
Group classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 ^a A-7-6
Sieve analysis percent passing											
No. 10	50 max										
No. 40	30 max	50 max	51 min								
No. 200	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No. 40											
Liquid limit, w_L				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plastic Index, I_P	6 max		NP	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min
Group Index ^b	0		0	0		4 max		8 max	12 max	16 max	20 max

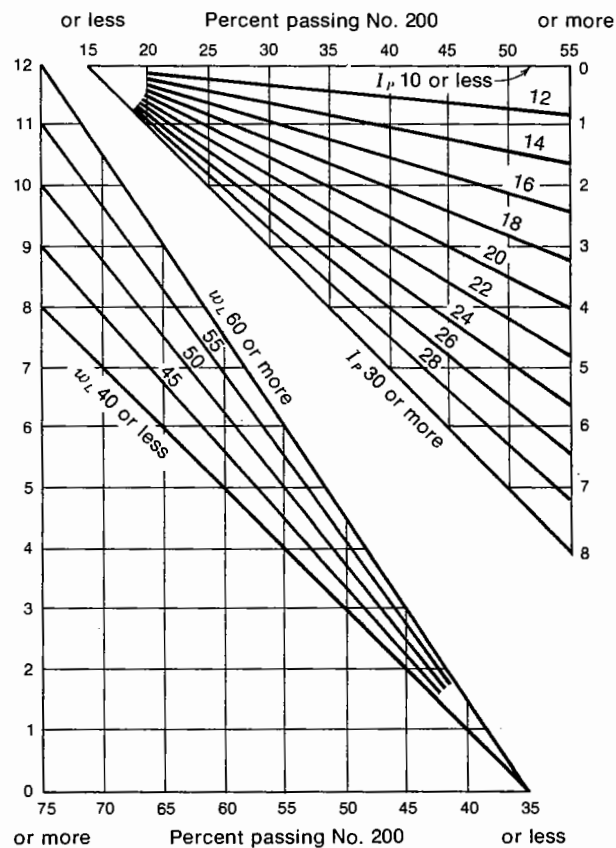
^a See Fig. 8-1b

^b Group index = $GI = 0.2a + 0.005ac + 0.01bd$ (See Fig. 8-1c)

(a) AASHTO soil classification system.



(b) Liquid limit and plasticity index ranges for A-4, A-5, A-6, and A-7 soil groups.



(c) Chart to obtain group index of a soil. Group index equals the sum of readings on both vertical scales.

Figure 8-1
Charts for use in the AASHTO Soil Classification system.

A-2 subgroup designation is obtained if the soil is an A-2 from the sieve analysis but has the plastic (w_L and w_P) properties of the A-4, A-5, A-6, or A-7 soil. Figure 8-1a is used by proceeding from left to right until the *first soil type is found* which has the properties of the soil under consideration. Soil type A-8 (not shown) is classified visually as peat or muck.

The AASHTO classification system also uses a group index rating to compare different soils in the same group. The group index is computed according to Eq. (8-1) or from Fig. 8-1c. The index thus found is rounded to the nearest whole number and placed in parentheses after the classification number as

A-2-6(3)

thus classifying the soil as an A-2 with the plastic properties of the A-6 and with a group index rating of 3. The group index GI is an AASHTO modification to the original BPR classification¹ to, as indicated earlier, rate a soil within its group classification. A soil with a smaller group index, all other factors being equal, should have a better road performance than a soil with a larger GI within the same group; e.g., an A-2-6(2) should be a better soil to use than an A-2-6(4).

The equation to establish the group index number is

$$GI = 0.2a + 0.005ac + 0.01bd \quad (8-1)$$

where a = percent passing the No. 200 sieve greater than 35 percent but not more than 75 percent, expressed as a positive whole number ($1 \leq a \leq 40$)

b = percent passing the No. 200 sieve greater than 15 percent but not more than 55 percent, expressed as a positive whole number ($1 \leq b \leq 40$)

c = portion of liquid limit greater than 40 but not more than 60, expressed as a positive whole number ($1 \leq c \leq 20$)

d = portion of the plasticity index greater than 10 but not exceeding 30, expressed as a positive whole number ($1 \leq d \leq 20$)

Since this is a linear equation, it can be seen that the upper part of Fig. 8-1c solves the term $0.01bd$ and the lower part of the figure solves $0.2a + 0.005ac$ of Eq. (8-1). Thus the sum of the values obtained from the two parts of the chart rounded off to the nearest whole number the group index GI of the soil.

To illustrate the method of soil classification by the AASHTO procedure, the following three soils will be classified:

Sieve No.	SOIL, % PASSING		
	A	B	C
4	—	—	69.3
10	68.5	79.5	59.1
20	—	—	48.3
40	36.1	69.0	38.5
60	—	—	28.4
100	—	—	19.8
200	21.9	54.3	5.1
Plastic properties			
$w_L =$	34.1	53.5	Nonplastic (NP)
$w_P =$	16.5	31.6	
Visual description	Light brown sandy and silty clay	Dark brown silty clay, trace of gravel	Medium brown very gravelly coarse sand

¹The original Bureau of Public Roads (BPR) classification was introduced in 1929. The current AASHTO designation was introduced about 1945.

Classifying soil A:

1. Compute the plasticity index, $I_p = w_L - w_p$

$$I_p = 34.1 - 16.5 = 17.6 > 10$$

Therefore, the value of $I_p > 11$ minimum controls.

2. Since 21.9 percent passes the No. 200 sieve, the soil is an A-2 with subgroup to be determined from I_p and w_L .
3. Proceeding across Fig. 8-1a from left to right with

$$w_L = 34.1 < 40 \text{ and } I_p = 17.6 > 11$$

the first soil which satisfies these criteria is an A-2-6 soil.

4. Next compute the group index GI which should not be more than 4 for any A-2 soil. For this soil we will both compute the group index and obtain the value from Fig. 8-1c. Computing, and using Eq. (8-1) for terms:

$$a = 0 \text{ since less than 35 percent passes the No. 200 sieve}$$

$$b = 21.9 - 15.0 = 6.9 \text{ (defined in Eq. (8-1))}$$

$$c = 0 \text{ since } w_L \text{ is less than 40}$$

$$d = I_p - 10 = 17.6 - 10 = 7.6$$

$$GI = 0.2a + 0.005ac + 0.01bd$$

$$GI = 0.2(0) + 0.005(0)(6.9) + 0.01(6.9)(7.6) = 0.52$$

and rounding to the nearest integer, obtain $GI = 1$

From Fig. 8-1c, the GI is approximately 0.6 from the upper chart and 0 from the lower chart, summing:

$$GI = 0.6 + 0 = 1$$

The final classification of soil A is:

Light brown, sandy, and silty clay, A-2-6(1)

Classifying soil B:

1. The plasticity index is

$$I_p = 53.5 - 31.6 = 21.9 > 10, \text{ therefore, 11 minimum controls}$$

2. The percent passing the No. 200 sieve is $54.3 > 35$, therefore, the soil is an A-4, A-5, A-6, or A-7.

3. With $w_L = 53.5$ and $I_p = 21.9$ the soil is an A-7, but we must still find if an A-7-5 or A-7-6. Inspection of Fig. 8-1b displays that the scale is too small to use, so we will compute the coordinates:

$$I_p = 21.9$$

$$w_L - 30 = 53.5 - 30 = 23.5 = I_p \text{ of line at } w_L = 53.5\%$$

Since $21.9 < 23.5$, the coordinates plot in the A-7-5 zone, and thus, the soil is an A-7-5. We must next compute the group index using Fig. 8-1c.

4. From Fig. 8-1c obtain 5.2 from the lower chart and 4.3 from the upper chart for a group index of

$$GI = 5.2 + 4.3 = 9.5, \text{ use 10}$$

The final classification of soil *B* is:

Dark brown, silty clay, trace of gravel, A-7-5(10)

Classifying soil *C*:

1. For soil *C* we find by a rapid elimination that the soil is either A-1 or A-3. With the percent passing the No. 40 sieve of $38.5 < 51$ but $38.5 > 30$ the soil must be A-1b. There is no group index for this soil.

The final classification of soil *C* is:

Medium brown, very gravelly, coarse sand, A-1b

B. UNIFIED SOIL CLASSIFICATION

The essential elements of this system of classifying soils were first proposed by Casagrande (1942) and were subsequently adopted by the U.S. Corps of Engineers for airfield construction. Currently, this system is used with slight modifications in many countries outside the United States. The system is rather widely used inside the United States by organizations such as the Corps of Engineers, Bureau of Reclamation, and with slight modifications by most consulting firms.

Table 8-1 presents the factors to consider in classifying a soil according to the Unified Soil Classification system. Basically a soil is:

Coarse-grained if more than 50% is retained		No. 200 0.075 mm	Fine-grained if more than 50% passes
Gravel	Sand		Silt or Clay
If more than 50% of coarse fraction is retained on the No. 4 sieve	If more than 50% of coarse fraction passes the No. 4 sieve		Fine-grained soil is: Silt (M) Clay (C) Organic (O)

We shall consider these subdivisions in more detail in the following:

1. Gravels or sands are

GW, GP or SW or SP

if less than 5 percent passes the No. 200 sieve; G = gravel; S = sand; W = well-graded; P = poorly graded. The well- or poorly graded designations depend on C_u and C_c as defined in Experiment No. 5 and with numerical values as shown in Table 8-1.

2. Gravels and sands are

GM, GC or SM or SC

if more than 12 percent passes the No. 200 sieve; M = silt; C = clay. The silt or clay designation is determined by performing the liquid and plastic limits on the (–) No. 40 fraction and using the A chart of Fig. 8-2. This chart is also a Casagrande contribution to the system, and the A line shown on the chart is sometimes called *Casagrande's A line*. The chart as presented in this manual has been modified based on Corps of Engineers findings that no soil (to date) has been found with coordinates which lie above the "upper-limit" line shown.

Table 8-1 The Unified Soil Classification System

Major divisions		Group symbol	Typical names		Classification criteria for coarse-grained soils		
Coarse-grained soils (more than half of material is larger than No. 200)	Gravels (more than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	$C_u = D_{60}/D_{10} > 4$ $C_c = 1 < D_{30}^2/D_{10} \times D_{60} < 3$		
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines	Not meeting all gradation requirements for GW		
		Gravels with fines (appreciable amount of fines)	GM	$\frac{d}{u}$	Silty gravels, gravel-sand-silt mixtures	Atterberg limits below A line or $I_p < 4$	Above A line with $4 < I_p < 7$ are borderline cases requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures	Atterberg limits above A line with $I_p > 7$		
	Sands (more than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines	$C_u = D_{60}/D_{10} > 6$ $C_c = 1 < D_{30}^2/D_{10} \times D_{60} < 3$		
			SP	Poorly graded sands, gravelly sands, little or no fines	Not meeting all gradation requirements for SW		
		Sands with fines (appreciable amount of fines)	SM	$\frac{d}{u}$	Silty sands, sand-silt mixtures	Atterberg limits below A line or $I_p < 4$	Limits plotting in hatched zone with $4 \leq I_p \leq 7$ are borderline cases requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures	Atterberg limits above A line with $I_p > 7$		
Fine-grained soils (more than half of material is smaller than No. 200)	Silts and clays (liquid limit < 50)	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity	1. Determine percentages of sand and gravel from grain-size curve. 2. Depending on percentages of fines (fraction smaller than 200 sieve size), coarse-grained soils are classified as follows: Less than 5%—GW, GP, SW, SP, More than 12%—GM, GC, SM, SC 5 to 12%—Borderline cases requiring dual symbols			
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays				
		OL	Organic silts and organic silty clays of low plasticity				
	Silts and clays (liquid limit > 50)	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts				
		CH	Inorganic clays of high plasticity, fat clays				
		OH	Organic clays of medium to high plasticity, organic silts				
	Highly organic soils	Pt	Peat and other highly organic soils				

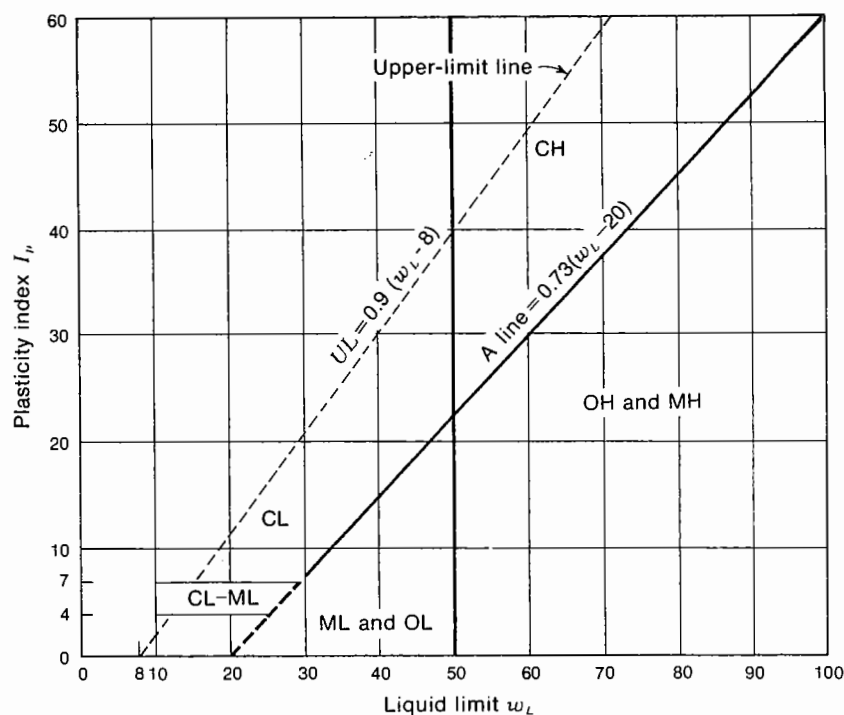


Figure 8-2
Plasticity chart to use with Table 8-1 for Unified Soil Classification.

3. Gravels and sands are

GW-GC	SW-SC	GP-GC	SP-SC
GW-GM	SW-SM	GP-GM	SP-SM

if between 5 and 12 percent of the material passes the No. 200 sieve.

4. Fine-grained soils (more than 50 percent passes the No. 200 sieve) are:

ML, OL, or CL

if the liquid limits are less than 50 percent; M = silt; O = organic soils; C = clay.

5. Fine-grained soils are:

MH, OH, or CH

if the liquid limits are greater than 50 percent; H = higher than 50 percent.

The liquid and plastic limits are performed on the (–) No. 40 sieve fraction of all the soils, including gravels, sands, and the fine-grained soils, using the procedures of Experiment No. 3. These limits are used with the plasticity (A chart) chart to determine the letter prefix M, O, or C, depending on the plotted location of the plasticity coordinates on the chart.

A visual description of the soil should be used with the Unified symbols to complete the classification as with the AASHTO classification system.

As an exercise we will reclassify the three soils previously classified in the AASHTO classification system.

Classifying soil A:

1. We have $21.9 < 50$ percent passing the No. 200 sieve and more than 50 percent passing the No. 4 sieve (since 68.5 percent passed the No. 10), therefore, the soil is either

SM or SC

2. Based on a w_L of 34.1 and $I_p = 17.6$ ($w_p = 16.5$), we obtain from the coordinates on the plasticity chart (Fig. 8-2) that the soil is a CL. Taking C for "clay" the soil is classified as

Light brown, sandy and silty clay, SC

Classifying soil B:

1. Since 54.3 percent passes the No. 200 sieve, the soil is immediately fine-grained and is MH, OH, or CH because the $w_L = 53.5 > 50$ percent.
2. From the plasticity chart at $w_L = 53.5$ and $I_p = 21.9$, the soil coordinates are difficult to read so we will compute $I_p = 0.73(w_L - 20)$ to obtain

$$I_p = 0.73(53.5 - 20) = 24.4 > 21.9$$

therefore, the soil is an MH from the coordinate location and the soil is classified as:

Dark brown clayey silt with trace of gravel, MH (note that the visual description has been slightly changed to fit the M-soil)

Classifying soil C:

1. With 5.1 percent passing the No. 200 sieve, soil is either GW, GP, SW, or SP (using 5 percent as sufficiently accurate and is strictly a matter of judgement).
2. It will be necessary to plot a grain size distribution curve and obtain

$$D_{60} = 2.00 \text{ mm}$$

$$D_{30} = 0.29$$

$D_{10} = 0.086$ (the reader should verify these values within plotting precision) and compute:

$$C_u = 2.00/0.086 = 23.3 > 6$$

$$C_c = 0.29^2/(2.00)(0.086) = 0.5 < 1 \text{ (not between 1 and 3)}$$

Since the gradation criteria are not met for well-graded, the soil is poorly graded (P suffix).

3. With 94.9 percent retained on the No. 200 sieve and with 69.3 percent passing the No. 4 sieve, the percent between the No. 4 and No. 200 is

69.3 - 5.1	=	64.2
% retained on No. 4 = 100 - 69.3	=	30.7
% passing No. 200	=	5.1
Total = 100.0 %		

With 64.2 percent as sand the soil is classified as

Medium brown, very gravelly, coarse, poorly graded sand, SP (note again the description has been changed slightly as more information becomes available)

Procedure *This is an Individual Project*

1. Each person will classify the soils assigned by the instructor¹ according to the Unified and/or AASHTO systems.
2. Give some thought to the information required of a particular classification system so that the amount of soil testing is kept to a minimum. Use the 1-point method of liquid-limit determination.
3. Be sure to obtain reasonably representative samples from the soil containers so that you can check your work if necessary.
4. Use the washing method to find the percent passing the No. 200 sieve, otherwise you may misclassify the soil. It is not unusual to dry-sieve a soil with considerable care and find only 40 to 45 percent passing the No. 200 sieve while with washing, well over 50 percent passes.
5. Instead of a formal report, submit this work in a folder with a flyleaf tabulation of results on the data sheets for this experiment and any additional data sheets used for sieve analysis, grain-size distribution curves, Atterberg limits, etc. Be sure to provide a visual description for each soil along with the classification system symbols.

¹The author normally requires six soils to be classified using two laboratory periods plus whatever additional time the student requires to do the necessary laboratory work/classification. This is done by keeping on hand 15 to 20 different appearing soils in containers holding 10 to 15 kg. Each student is given 5 numbers corresponding to containers + the regular laboratory soil. This tends to ensure individual work on the project.

Experiment No. 9

MOISTURE-DENSITY RELATIONSHIPS (Compaction Test)

References

- AASHTO T99-70 (standard) and T180-70 (modified)
ASTM D698-70 and D1557-70
Burmister, D. M., (1965), Environmental Factors in Soil Compaction, *ASTM STP no. 377*, pp. 47-66.
Gordon, B. B., W. D. Hammond, and R. K. Miller (1965), Effect of Rock Content on Compaction Characteristics of Clayey Soil, *ASTM STP no. 377*, pp. 31-46.
Johnson, A. W., and J. R. Sallberg (1962), "Factors Influencing Compaction Results," *Highway Research Board Bulletin no. 319*, 148 pages.
Lambe, T. W., (1960), Compacted Clay: A Symposium, *Trans, ASCE*, vol. 125, pp. 682-756 (also in *J. Soil Mech. Found. Div., SM 2*, May).
Lee, P. Y., and R. J. Suedkamp (1972), Characteristics of Irregularly Shaped Compaction Curves, *Highway Research Record no. 381*, pp 1-9.
Proctor, R. R., (1933), Fundamental Principles of Soil Compaction, *Eng. News-Record*, Aug. 31, Sept. 7, 21, and 28.
Seed, H. B., and C. K. Chan (1959), Structure and Strength Characteristics of Compacted Clays, *J. Soil Mech. Found. Div., ASCE, SM 5*, October, pp. 87-128.
Wilson, S. D., (1950), Small Compaction Apparatus Duplicates Field Results Closely, *Eng. News-Record*, Nov. 2, pp. 34-36.

Objective

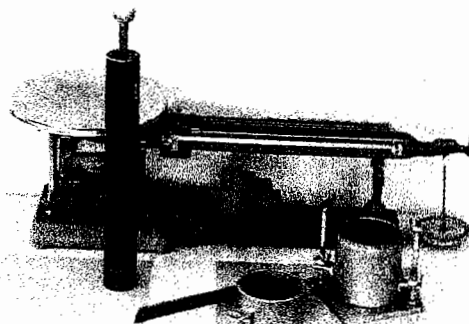
To familiarize the student with the laboratory compaction test and to obtain the moisture-density relationship for a given compactive effort on a particular soil.

Equipment¹

Compaction mold² with base plate and collar (refer to Fig. 9-1)

Figure 9-1

Essential compaction-test equipment: Large (20-kg) balances, compaction-mold base plate and collar, 24.5-N \times 0.305-m drop hammer, steel straightedge to smooth ends of compacted sample. Not shown is mixing pan (and/or mechanical mixer), optional 44.5-N hammer, graduated cylinder for water, and mixing spoon.



¹Instructor should obtain water content of soil for this experiment prior to period—also values of w_L and w_p from earlier experiments so that OMC can be estimated using Fig. 9-2.

²SI mold with dimensions of 10.3 cm diam \times 12.0 cm height and volume = 1000 cm³ can be made from commercially available stainless steel pipe (to avoid rust) using iron pipe size 4-in diam. Machine ID to 10.3 cm and weld two holddown lugs cut from a piece of 1 \times 1-in stainless steel angle. Turn the OD to fit the standard mold collar notch and use the base plate from the 1/30-ft³ mold. Turn the ID of the collar to match the ID of the mold.

Compaction hammer (24.5 N × 0.305 m drop or 44.5 N × 0.46 m drop) or mechanically operated hammer
 10 to 12 moisture cans
 Sample extruder (jack) or mechanical pulverizer as in Fig. 9-4
 Large mixing pan
 Soil mixer (optional)

General Discussion

In 1933, R. R. Proctor presented four articles in the *Engineering News-Record* (Proctor, 1933), which are the basis for the standard compaction test currently used (sometimes called Proctor tests or simply "Proctors").

The standard test consists of taking about 3 kg of soil, passing it through a No. 4 sieve¹, adding water, and compacting it into a 944 cm³ mold in three layers with 25 blows per layer, using a 24.5 N compaction hammer dropping 0.305 m onto the soil. This delivers a nominal compaction energy (kilojoules or kJ) to the soil of:

$$CE = \frac{3(25)(24.5)(0.305)}{9.44 \times 10^{-4}(1000)} = 593.7 \text{ kJ/m}^3$$

The compacted sample is broken down to the No. 4 sieve size *as determined visually*, water-content samples are taken, more water is added, the soil thoroughly remixed, and the process of compacting a mold of soil is repeated.² This sequence is repeated a sufficient number of times that a curve of dry density vs. water content can be drawn which has a zero slope (a maximum value) with sufficient points on either side of the maximum density point to adequately define its location. Dry density is always the ordinate of this curve. The maximum ordinate value is termed *maximum density*, and the water content at which this dry density occurs is termed the *optimum moisture content* (OMC).

Several comments can be made at this point:

1. For a test to be standard, the results should be reproducible, and whether one uses a fresh sample each time or breaks down the compacted sample and recompacts it, often yields different maximum dry-density values—differences of 0.8 to 1.2 kN/m³ not being uncommon values obtained in this manner.³
2. When testing air-dry soils, unless the first test sample is mixed with the first increment of water and allowed to cure (say, overnight), the results may yield a curve with erratic points (the locus of points is not a smooth curve) on the dry side of the optimum or peak point. Noncuring may also shift the OMC to the right (tend to increase it) on the density vs. water-content curve. Some soils may yield a somewhat erratic "dry side" curve as a soil characteristic (see Lee and Suedkamp, 1972).
3. If the cylinder of soil is not compacted in three approximately equal increments, the curve points will be somewhat erratic, i.e., will not fall on a smooth curve on either side of optimum.

¹Both AASHTO and ASTM include standard procedures for soils which contain materials larger than the No. 4 sieve.

²At the time of this revision (1977) ASTM appears to be changing the test procedure to use fresh soil samples for each test point; thus, for 5 points approximately 15 kg of soil is required. It is doubtful that this method will be very popular (or provide any advantage over the reused soil method) due to the large amount of successful data based on reusing the soil. One could always have used fresh samples but this was seldom done—one major factor is the huge quantity of soil required.

³Research by the author indicates that additional mechanical mixing, say, 8 to 10 min will reduce these differences to the order of perhaps 0.2 to 0.4 kN/m³. Differences are negligible (in most cases) for soils of low plasticity. Differences are less also for cured soils (mixed with water for 10-12 h before test).

4. To avoid large numbers of compacted cylinders and since about five well-placed points will define the curve very well, it is desirable to begin the first test at a water content about 4 to 5 percent from OMC. Then adding 2 percent moisture (by weight) on successive trials, the last point should be about 4 to 5 percent on the wet side of the curve peak. To assist in determining the approximate starting-point water content, Fig. 9-2 is included. This chart should give the OMC within ± 2 percent if the liquid- and plastic-limit values are reasonably accurate.

With practice, one may estimate OMC by adding water until a squeezed handful of soil is deformed with finger marks for about an average squeeze effort and the molded soil resists breaking apart. Dry the sample to obtain the water content as the OMC estimate.

5. The compaction mold should be placed on a surface which does not vibrate during the compaction process so that the hammer energy is not lost in producing foundation displacements. ASTM indicates that the mold should rest on a base consisting of a rigid cylinder or block of concrete with a mass of at least 90 kg (200 lb).

The original Proctor procedure has been modified somewhat by using a controlled hammer drop as "25 blows from a hammer dropping free from a height of" 0.305 m. Originally Proctor stated "... 25 firm 12-in strokes." The current method of data presentation uses a curve of dry density vs. water content (abscissa). The data seem to have been presented originally as void ratio vs. water content.

Soil compaction is generally the cheapest method of soil stabilization available. Soil stabilization is improving the undesirable physical properties of a soil to achieve the desired shear strength, structure, and void ratio. There are many methods of stabilizing soil using chemicals such as lime, lime-flyash admixtures, cement, and phosphoric acid compounds, but these methods are usually more expensive and may utilize compaction methods in addition to the admixture since its incorporation into the soil mass may disturb the soil considerably.

Compaction achieves soil stabilization through the input of energy into the soil by:

SIMULATED		
Method	In laboratory	In field
Impact	Standard compaction test	Nothing comparable
Kneading action	Harvard miniature apparatus, Hveem method	Sheepsfoot roller wobble wheel
Vibration	Vibratory table; see Experiment No. 18	Vibratory rollers and compactors
Static (or dynamic) compression	Compression machines	Smooth wheel rollers

Generally, the compaction effort imparts to the soil:

1. An increase in shear strength since this is a function of density (the other variables are structure, ϕ and c).
2. An increase in swell potential
3. An increase in density [$\gamma = f(e)$]
4. A decrease in shrinkage
5. A decrease in permeability [$k = f(e)$]
6. A decrease in compressibility [$S = f(e)$]

From this list of soil properties affected by compaction, it can be seen that there is more to the problem of specifying compaction than merely requiring com-

paction to increase the density. It is also important to consider side effects; fortunately, this is not as serious as might be expected at first in view of the method of stipulating compaction which is often used— X percent of standard compaction, or the modified AASHTO methods. This comes about in specifying the type of soil to be used, that is, limiting the type of soil to which compaction criteria are applied on a given project to eliminate, for example, volume-change problems.

It is now recognized that the structure of the resulting soil mass (especially when fine-grained soils are present) is intimately associated with the compaction process and the water content at which the mass is compacted. This concept is extremely important for compacting the clay cores of dams (as one example), where settlements could cause cracks in the core. It has been found that the *dispersed* soil structure obtained by compacting these soils on the wet side of optimum results in a soil which has a somewhat lower shear strength but which can undergo large deformations without failure (cracking) and the resulting large leakage and/or actual dam failure. Compaction of the soil on the wet side of optimum also reduces the permeability, as compared with compacting the soil on the dry side of optimum.

Conversely, the *flocculated* structure resulting from compacting a clayey soil on the dry side of optimum is less susceptible to shrinkage but is more susceptible to swell. The ultimate strength of soils with flocculated structures is higher at low strains than is the strength of soils with dispersed structure, i.e., the soil tends to *brittle* failure. The residual strength of the soil compacted on the dry side of optimum is nearly the same as the ultimate strength of soil compacted on the wet side of optimum. Thus, for highway work, where low strains beneath the pavement are desirable, the soil should be compacted at a water content from dry of optimum to optimum. The soil shell surrounding the clay core of an earth dam should be also compacted to produce a flocculated structure since strength is more important than seepage. The clay core, on the other hand, should be compacted to produce a dispersed structure since large settlements are possible and the soil would need to be able to tolerate these large strains without cracking with a resulting piping/seepage failure.

From this brief discussion, it is evident that compaction criteria should be based on a consideration of soil structure, strength, permeability, etc., as design properties needed rather than simply obtaining a compaction curve in the laboratory and requiring that the soil be compacted to some percent relative compaction although in many cases—especially where density (and settlement control) is the only needed property—this does produce (as it has in the past) a satisfactory product.

Relative compaction is the term used to compare the in situ compacted soil to the laboratory compaction curve. Relative compaction is defined as

$$\text{Relative compaction} = \frac{\text{density of compacted field soil}}{\text{maximum laboratory dry density}} \times 100 \text{ (percent)} \quad (9-1)$$

and may be more or less than 100 percent. For example, a particular project may specify that soil be compacted to 90 percent relative compaction; another project may specify 105 percent relative compaction. If the maximum dry unit weight is 18.50 kN/m^3 , the field unit weight is:

$$\text{At 90 percent relative compaction } 18.50(0.90) = 16.65 \text{ kN/m}^3$$

$$\text{At 105 percent relative compaction } 18.50(1.05) = 19.42 \text{ kN/m}^3$$

From the table of compaction methods given earlier it can be seen that there is no field compaction counterpart to the laboratory impact method of soil compaction. The reuse of the laboratory soil in developing the compaction curve results in

considerably more soil processing than given the field soil. Considerable data has been collected which indicates soil structure, density, and OMC depends on the method of providing the compaction energy (kneading, impact, vibration, etc.). In spite of this shortcoming, the standard compaction test as outlined here is widely used to establish the compaction criteria. There is a considerable collection of data that indicates that this method works; also since the time the test was proposed, no one has developed a better method so there is no valid reason at present to discard the procedure.

Just prior to and during World War II, heavy military aircraft began requiring a subgrade density beneath airfield runways greater than that provided by the standard compaction density. Rather than use relative compactions greater than 100 percent, a modified test was introduced (now termed *modified proctor*, *modified AASHTO*, or *modified compaction test*). This test is as follows:

Mold	944 cm ³
Hammer	44.5 N
Layers	5 at 25 blows/layer
Hammer fall	0.46 m

The test essentials are the same as the standard compaction test, i.e., use (–) No. 4 sieve material or consult ASTM or AASHTO for cases where larger gravel is present, develop the curve as for the standard test by adding increments of water, compact, break the sample down, and repeat the process until a curve is obtained.

The modified compaction test introduces a nominal compaction energy into the soil of 2710 kJ/m³ (the reader should verify this) or about 5 times the compaction energy of the standard test. For this extra energy an increase in density of 5 to 10 percent is obtained and a reduced value of OMC over the standard compaction test. The data for the modified compaction test are also presented as a curve of dry density vs. water content.

The plot of dry density vs. water content indicates that the process of compaction at *any given compactive effort* becomes more efficient up to a certain water content (OMC), then the efficiency decreases. Efficiency increases as water is added initially causing a slaking and/or general breakdown of interparticle bonds of the clay lumps and possibly some lubrication. On the wet side of optimum the lumps have generally been slaked and the excess moisture causes high instantaneous pore pressures as the hammer impacts. The high pore pressures result in a greatly reduced shear strength and the hammer simply shears (remolds) the soil rather than increasing the density further.

The soil mass involved in the compaction process starts as a three-phase system of soil, water, and air. During the initial trials considerable air is present, but the process results in a change of state with more soil and water being present. Even at OMC there is considerable air present. On the wet side of optimum, the principal effect is to displace more and more air with water. If the process were completely efficient, it might be possible to replace all the air in the voids with water to produce a two-phase system (a condition of *zero-air-voids*). Since it is never possible to get all the air out of the voids, which would result in a condition of $S = 100$ percent, any compaction curve *will always fall below* the zero-air-voids curve.

For any given water content w the zero-air-voids dry unit weight γ_{zav} is computed as

$$\gamma_{zav} = \frac{G_s \gamma_w}{1 + w G_s} \quad (9-2)$$

The locus of points produces a slightly concave upward curve.

Numerous factors influence soil compaction including:

Temperature

Size of molds (hold the diam/ht ratio approximately constant and effect is negligible)

Distribution of blows on any layer

Excess quantity of soil in mold

Type of soil (Note only cohesive soil can be compacted using the impact method)

Amount of processing (mixing, curing, manipulation)

The reader should use the reference list for additional study of the factors which can influence compaction results.

Procedure *This Is a Group Project*

Note: A water-content sample should be taken 24 h prior to this test so that the initial water content can be reasonably estimated; otherwise it may take 6 to 8 trials to obtain the compaction curve—especially for any soils where OMC is 17 to 22 percent.

1. Each group should take 3 kg (nominal weight) of air-dry soil, pulverize sufficiently to run through the No. 4 sieve, then mix with the initial percentage of water based on estimated dry weight. The initial percentage of water should be based on the present water content, a desired initial moisture content 4 to 5 percent below OMC, and obtain OMC from Fig. 9-2 or other means of estimation.

The soil and water should be premixed and cured for about 24 h prior to the test, but for student laboratories this may be omitted.

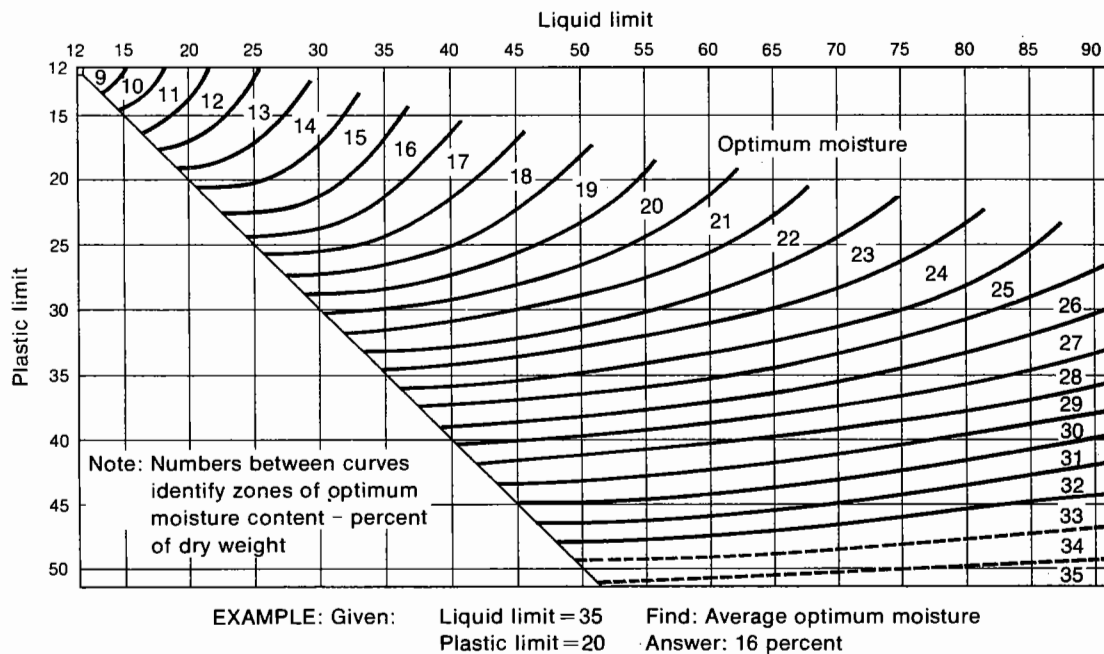


Figure 9-2

Chart to find the approximate optimum moisture content (OMC) of a soil for use in the standard compaction test. (This chart is reproduced by permission from Johnson and Sallberg (1962), p. 125.)

2. If the soil has been "cured," add 1 percent moisture by weight [$.01 \times 3 = 0.03$ kg (30 ml)] to account for evaporation losses. Mix this water into the soil carefully.
3. Weigh the compaction mold but do not include collar or base plate.
4. Measure the compaction mold to determine its volume [or at the discretion of the instructor assume the volume is 944 cm^3 (or 1000 as appropriate)].
5. Use either the standard or modified compaction method as specified by the instructor and compact a cylinder of soil. If you use a 1000-cm^3 mold, use 26 blows/layer instead of 25 to produce the same compaction energy and for either test.
6. Carefully strike both the top *and base* of the compacted cylinder of soil with a steel straightedge. Fill in any holes in the compacted specimen with either soil or gravel which is smaller than the hole where the smoothing process removes any pebbles or soil.

Note: If the mold is not filled above the collar joint from the last compacted layer, do not add soil to make up the deficiency—redo the test. You can avoid this unpleasant situation, however, by carefully watching and, after about 10 blows on the last layer, if the soil is below the collar joint, adding enough material to fill above the collar joint and then continuing with the remainder of the blows. You should try not to have more than about 6 mm of soil above the collar joint on the other extreme (see Fig. 9-3). If you have much more than this amount of excess and are not careful, you will remove the last layer of compacted soil cake when you remove the collar. If you do this, redo the test since you can never replace the soil cake properly. If the collar is hard to remove, do not risk twisting off the third (or last) layer of soil; take a spatula and trim along the sides of the collar until it comes off easily. Remember that you have an error multiplier of 1060 in this project; therefore, an error of 15 g of soil is about 0.15 kN/m^3 of compaction error—and 15 g of soil is not a very large quantity.

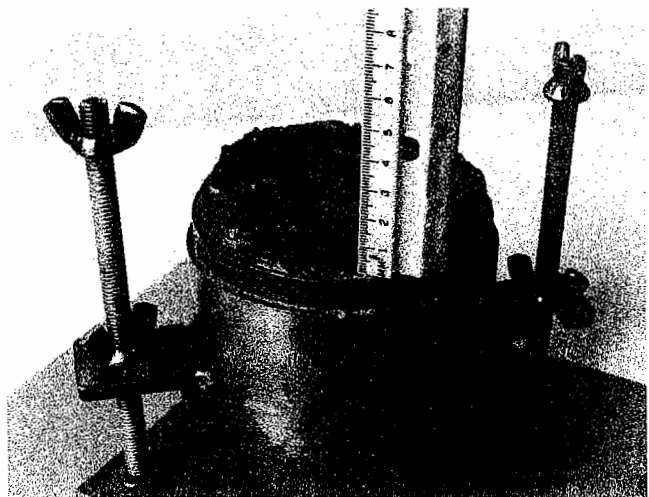


Figure 9-3

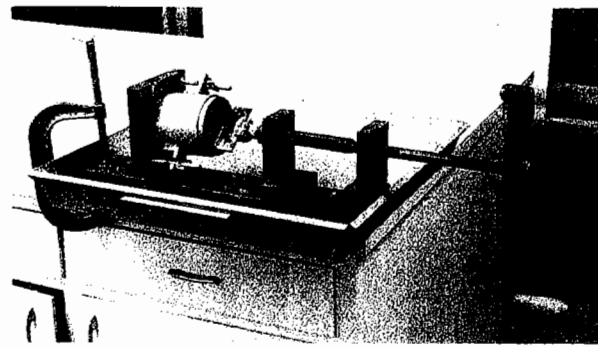
Compacted soil specimen with collar removed and initial trimming. Note that soil projects only about 6 mm above mold.

7. Weigh the mold and cylinder of soil.
8. Extrude the cylinder of soil from the mold, split it, and take two water-content samples—one near the top and the other near the bottom—of as much as the moisture cups will hold (about 100 g). Alternatively, use the sample reducer¹

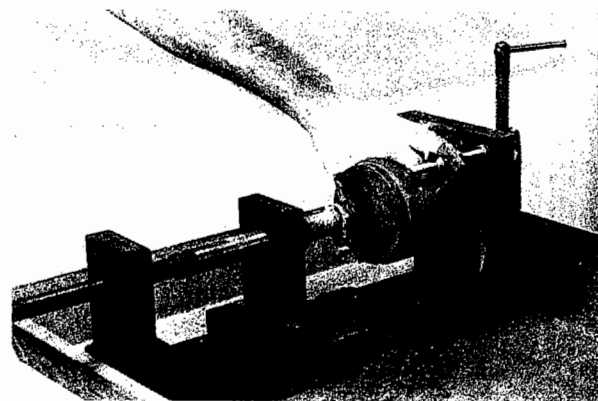
¹This device was developed and patented by the author and has been successfully used for several years at considerable savings in time (often 2 to 3 h per test).

Figure 9-4

A compaction sample pulverizer developed by author.
(a) Pulverizer with sample attached for augering from mold.



(b) Pulverizing a compacted sample. Note the soil is reduced to approximately No. 4 sieve size (visual inspection).



(Fig. 9-4) to remove the soil and pulverize it directly from the mold. Again take two water-content samples near the top and bottom of the sample as the soil emerges from the mold.

When using the sample ejector (jack), it may be found that some molds taper slightly so that the bottom is larger than the top. If this is the case, the sample will extrude much more easily if the large end exits the sample.¹

9. Break the sample to (—) No. 4 sieve size (unless the sample reducer device was used) and add 2 percent (based on the original sample weight of 3 kg) of water. Carefully remix and repeat steps 5 to 9 until, based on wet weights, a peak value is followed by two slightly lesser compacted weights.
10. Return to the laboratory the following day and weigh the oven-dry water-content samples to find the actual average water content of each test.
11. Compute the dry unit weight and make a plot of γ_{dry} vs. water content, with γ_{dry} as the ordinate (see Fig. 9-5). On this curve, plot the zero-air-voids curve.

If G_s is not known, assume that the zero-air-voids density at optimum moisture is 5 percent larger than the maximum dry density. From this assumption you can compute a value of G_s for the soil and find the dry density at other values of water content to plot the zero-air-voids curve. If the curve falls below the compaction curve at any point, the 5 percent assumption was not correct, so add 1 percent increments of density and recompute until you obtain a zero-air-voids curve which everywhere falls above the compaction curve. Be sure to use a good scale for the compaction curve since this plot is used for quantitative purposes.

12. Show the liquid limit, plastic limit, G_s , and the AASHTO classification of this soil on the data sheet beside the compaction curve plot. If you used a different

¹If you use a fresh sample for each point, discard the remainder of the compacted sample and repeat steps 5 through 8.

COMPACTION TEST

Data Sheet 10

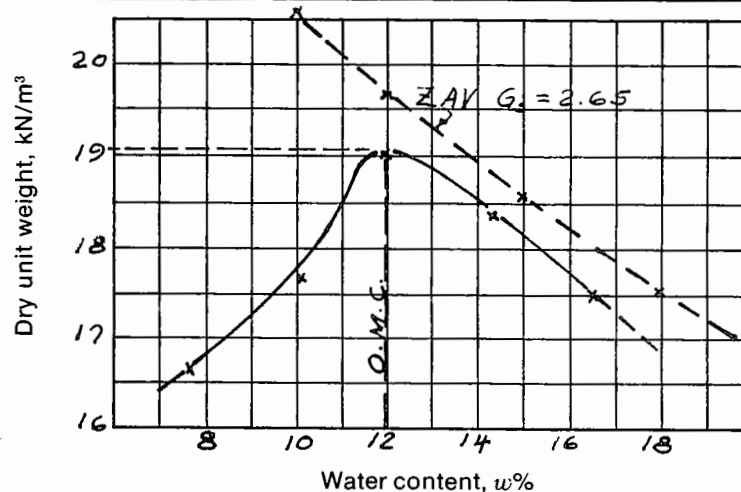
Project Standard Compaction test Job No. —
 Location of Project Bradley University Boring No. ~ Sample No. ~
 Description of Soil Light Br. Silty, Sandy Clay A-2-6(3); $w_L = 28.3$; $w_p = 14.1\%$
 Test Performed By JEB & M.C. Date of Test 10-8-76
 Blows/Layer 26/3 No. of Layers 3 Wt. of Hammer 24.5 N
 Mold dimensions: Diam. 10.3 cm Ht. 12.0 cm Vol. 1000 cm³

Water Content Determination

Sample no.	1		2		3		4		5		6	
Moisture can no.	75	77	81	86	87	88	93	104	109	110		
Wt. of can + wet soil	93.80	98.24	93.96	101.76	97.62	88.47	95.22	102.53	100.65	97.07		
Wt. of can + dry soil	88.64	92.66	87.37	94.46	89.00	80.94	85.47	91.83	88.83	85.68		
Wt. of water	5.16	5.58	6.59	7.30	8.62	7.53	9.75	10.70	11.82	11.39		
Wt. of can	22.57	22.02	22.00	22.32	17.85	17.80	17.32	17.46	17.21	17.25		
Wt. of dry soil	66.07	70.64	65.37	72.14	71.15	63.14	68.15	74.37	71.62	68.43		
Water content, w%	7.81	7.90	10.08	10.12	12.11	11.93	14.31	14.39	16.51	16.64		

Unit Weight Determination

Assumed water content	8	10	12	14	16	
Water content, w%	7.8	10.1	12.0	14.4	16.6	
Wt. of soil + mold	3757.2	3921.4	4109.0	4082.5	4015.2	
Wt. of mold	1933.0					
Wt. of soil in mold, g	1824.2	1988.4	2176.0	2149.5	2082.2	
Wet unit wt., kN/m ³	17.89	19.50	21.34	21.08	20.42	
Dry unit wt., kN/m ³	16.60	17.71	19.05	18.43	17.51	



Optimum moisture = 12.0 % Maximum dry unit weight = 19.05 kN/m³

Figure 9-5

Data from a compaction test including dry unit weight vs. water content to obtain maximum dry unit weight and OMC.

Experiment Nine

soil for this test than was used earlier for Experiment No. 3, 5, and 7, you should perform these tests as part of this project.

13. In your sample computations, show the derivation of Eq. (9-2) for zero-air-voids.
14. (*Optional*) Based on the value of specific gravity G_s , from step 11 or test value, compute the void ratio of the soil for each test point. On a sheet of graph paper from the data sheet section, replot the compaction curve, then, using void ratio for the ordinate (on right side) plot e vs. water content (use same water-content scale for both) and comment on which method displays the data best. Proctor's original method used e vs. water content.
15. (*Optional*) State what you recommend as relative compaction and the compaction moisture content for this soil in a 10-m highway fill.

Experiment No. 10

DETERMINATION OF IN-PLACE SOIL DENSITY

References

- AASHTO T181-62 (Method with Large Sizes of Coarse Aggregate)
T191-61 (Sand Cone Method)
T205-64 (Balloon Density)
- ASTM D1556-64 (Sand Cone) and D2167-66 (Rubber Balloon Method)

Objective

To present to the student two commonly used methods to determine the density of soil in situ.

Equipment

- Sand-cone apparatus (see Fig. 10-1)
- Balloon-density apparatus
- Digging tools
- Friction-top cans [4000 ml (1 gal)]¹
- Sack per group to salvage density sand

General Discussion

Once compaction criteria are established for the soil to be used at a particular site, generally with both moisture and density limitations, some means of verification of the results must be used. On all small projects and nearly all large projects, this verification is achieved by either the sand-cone method or the balloon-density method (see Fig. 10-2). On a few large projects, nuclear devices have been and are being used. The nuclear method is beyond the scope of this manual and will not be considered further.

Basically, both the sand-cone and balloon-density methods use the same principle. That is, one obtains a known weight of damp (or wet) soil from a small excavation of somewhat irregular shape (a hole) in the ground. If one knows the volume of the hole, the wet density is simply computed as

$$\gamma_{\text{wet}} = \frac{\text{weight of damp soil}}{\text{volume of hole}} \quad (10-1)$$

and if one obtains the water content w of the excavated material, the dry unit weight of the material is

$$\gamma_{\text{dry}} = \frac{\gamma_{\text{wet}}}{1 + w} \quad (10-2)$$

The sand-cone and balloon methods enable one to find the volume of the hole in Eq. (10-1). With the balloon method, the volume is found as a direct measurement of the volume of water pumped into a rubber balloon which fills the hole. This volume is read directly from a graduated cylinder which forms the reservoir

¹The author has found that standard 3-lb coffee cans with plastic lids make the best field-sample cans. They hold approximately 3000 ml and are easier to open, close, and empty. The plastic lid fits tight enough that no moisture is lost over a period of several days.

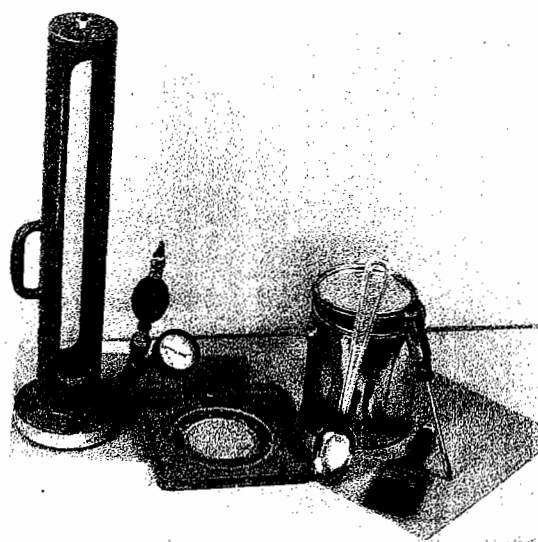
Figure 10-1

In-place soil-density equipment.

(a) Sand-cone equipment: Sand cone, plastic jug, hole template, paint brush to collect soil from template, spoon and screwdriver for digging, and 3-lb coffee can with plastic lid (painted) for collecting field soil.



(b) Balloon density equipment: Balloon device equipped with pressure gage (optional), hole template, digging tools, paint brush for collecting soil from template, friction-top can for collecting field soil.



for the balloon. This rapid means of finding the volume of the hole is often a distinct advantage timewise over the sand-cone method, which is more indirect. Care must be taken not to leave a hole that is too irregular in shape or with sharp projections which can rupture the balloon. A zero reading must be obtained prior to using the device, after which, unless excessive evaporation occurs in the reservoir or the balloon ruptures, many hole-volume readings can be taken before a new zero reading is required.

The sand-cone method is an indirect means of obtaining the volume of the hole. The sand used (often Ottawa sand) is generally material passing the No. 20 sieve but retained on the No. 30 sieve.¹ Although (–) No. 30 and (+) No. 40 or (–) No. 30 and (+) No. 50 sieve material can be used, it is generally desirable to have a uniform or “one-size” sand to avoid segregation problems (a volume of fine sand may weigh more than the same volume of coarse sand, and a volume of the mixture

¹It should be noted that ASTM D1556-64 requires only No. 200 < sand < No. 10.

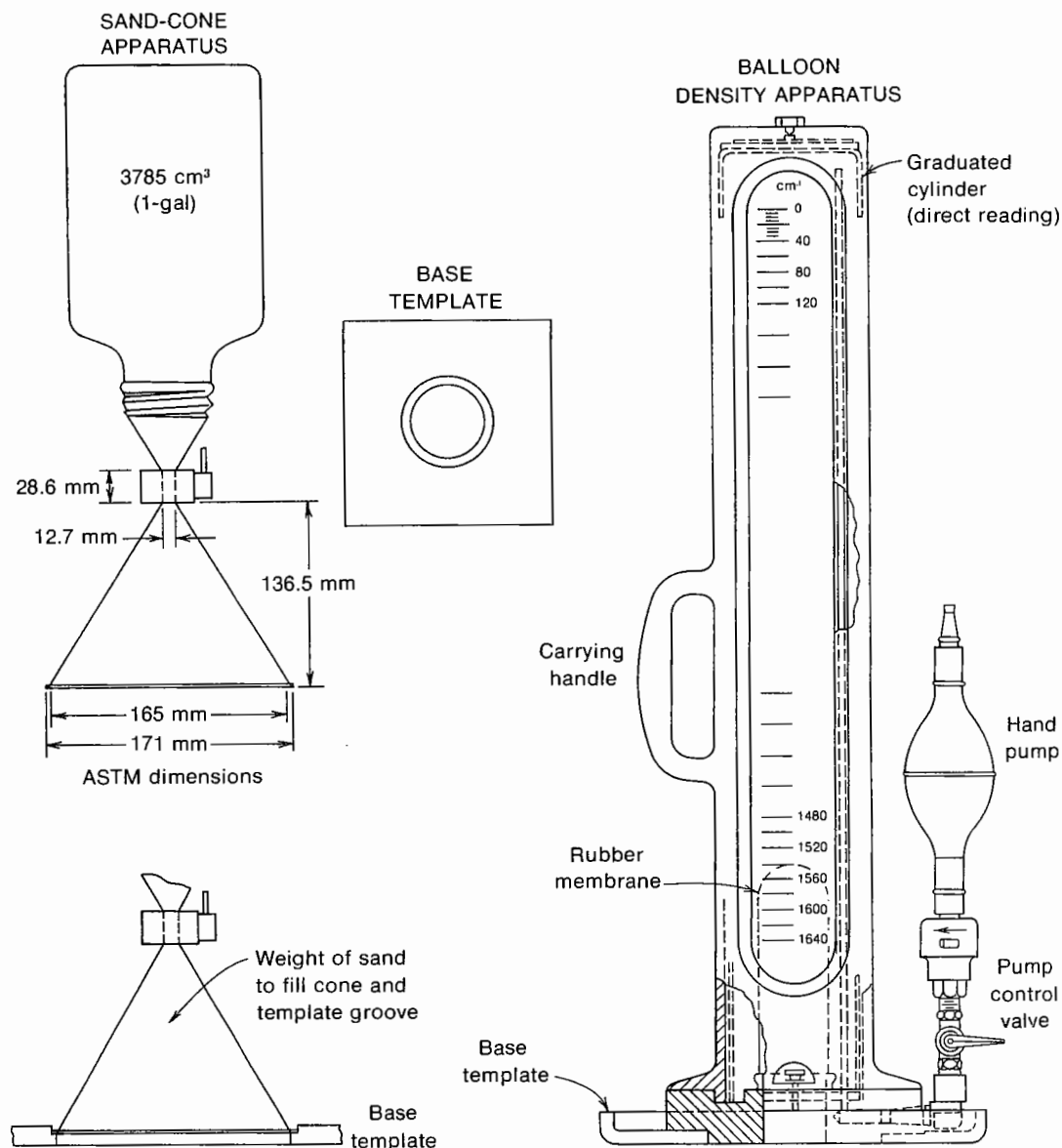


Figure 10-2
Typical details of sand-cone and balloon density apparatus.

may weigh still more), so that under the same pouring conditions, about the same soil structure (*of the same density*) can be obtained and duplicated as required.

If one has a constant-density material of, say, 1.60 g/cm^3 and pours 4800 g of this material into an irregular-shaped hole, the volume of the hole can be found by proportion¹ as:

$$\frac{V}{4800} = \frac{1 \text{ cm}^3}{1.60 \text{ g/cm}^3}$$

$$\text{or } V = \frac{4800}{1.60} = 3000 \text{ cm}^3$$

¹It is a decided computational convenience to use g/cm^3 , g, and cm^3 in these computations even though they are not strictly correct SI units.

and in general, the hole volume is

$$V_{\text{hole}} = \frac{\text{weight of material used to fill hole}}{\text{unit weight of material}} \quad (10-3)$$

Several comments are in order at this point:

1. The balloon-density apparatus is available in only two sizes:¹

1596 cm³ and 2230 cm³

The most commonly used sand-cone apparatus uses a 3785 cm³ (1-gal) glass or plastic jug with sufficient material to fill a hole (and cone combined) of not over 3800 cm³—and depends on how carefully the jug is filled prior to use. Two gal (7500 cm³) and $\frac{1}{2}$ gal (1900 cm³) jugs are also available, but not widely used.

In general, the field test holes will be quite small, thus the error multiplier is large, and it is absolutely essential that no soil be lost during excavation, or the volume determination be done in any way which gives an apparent hole volume that is too large.

As a guide, ASTM suggests that for reasonable results the test be as follows:

Max. sieve size of soil	Vol. of test hole (cm ³)	Size of water-content sample (g)
No. 4	700	100
12.7 mm	1400	250
25.0	2100	500
50.0	2800	1000

In laboratories where oven space is available, the best water-content results are obtained by drying the entire amount of soil excavated from the test hole.

2. The excavation should be as rapid as possible to maintain a representative moisture content of the natural soil in the friction-top can. As soon as the excavation is complete, the can should be sealed.
3. When using the sand-cone method, avoid vibrating either the ground in the area or the gallon jug as this will introduce too much sand into the hole and thus increase the apparent volume of the hole.

Procedure *This Will Be a Group and Class Project*

1. Each group will perform a sand-cone density test in an area designated by the instructor.
2. Each laboratory group will (as a joint project) test a single hole using the balloon apparatus to measure the volume of the hole.
3. The entire amount of soil collected from the test holes should be oven-dried to determine the water content of the in-place soil.
4. Use data sheets from the data sheet section of manual for project.

A. FIELD WORK

The field work will be done first as follows:

1. Prior to going to the field: (a) Each group will weigh the sand-cone jug full of sand to obtain W_j and the friction-top can with lid to obtain W_c . (b) Each group will place the balloon-density apparatus *on its template* on a flat surface and ob-

¹SOILTest, Inc., Evanston, IL, sells balloon cylinders in metric calibration to replace the fps graduated cylinders.

tain a zero reading. Place the zero reading on the blackboard so it can be averaged for the class.

2. Each group will then proceed to the field site and excavate a hole using the template provided with the sand cone (see Fig. 10-3). Carefully place all soil removed from the hole into the friction-top can and tightly seal. Before placing the template on the site, be sure to carefully smooth the excavation zone.
3. Next, with the valve closed, turn the sand cone upside down onto the template and open the valve. When the sand ceases to pour, close the valve and lift off. Salvage as much of the sand (Ottawa sand is relatively expensive) from the hole and template as possible and place in the sacks provided.
4. One group, as designated by the instructor, will place the balloon template at a site and excavate a hole identical to that for the sand-cone method but of not nearly as large a volume.
5. Each group will then place the balloon apparatus on the template, pump the balloon into the hole for a volume measurement, and record the final reading of the water reservoir. If the device is equipped with a pressure gage, use a pressure of not over 0.2 kg/cm^2 (3 psi).
6. For on-campus student work, be sure to landscape your hole so that the site is left as neat as you found it.
7. This completes the field work. Return to the soil laboratory. Check to make sure you are returning with all the equipment you brought out.

B. IMMEDIATE LABORATORY WORK

1. Each group will weigh the can of damp soil obtained from the field and record on the data sheet in the appropriate line. Pour the soil into a weighed container and place in the oven for drying.
2. Each group will weigh its partially empty sand-cone jug and record on the appropriate line of the data sheet.
3. Place the balloon volume readings on the blackboard in the soil laboratory, average, and subtract the averaged zero readings for a class average of the hole volume. Record the averaged final reading and the initial readings on the data sheet together with the averaged hole volume V_h .
4. The group taking the soil sample from the balloon test hole will put the wet weight of the soil from that test on the blackboard.

C. LATER LABORATORY WORK

1. Return to the laboratory the next day and weigh the oven-dry samples to find the dry weight of soil removed from the holes. Record this in the appropriate location on the data sheets.
2. Each individual will compute the water content and the wet and dry unit weights of the test hole of the individual's own group.¹
3. Record on the blackboard for the rest of the laboratory group the weight of the dry soil for the balloon test hole.
4. Each individual will compute the water content and the wet and dry unit weight of the balloon test hole.¹

D. CALIBRATION OF EQUIPMENT

Each group must determine three items of information:

1. The density (g/cm^3) of the sand used in the field density test
2. The weight of sand required to fill the sand cone and template groove (see Fig. 10-2)
3. The calibration of the balloon-density apparatus.

¹After performing Procedure D following.

Figure 10-3

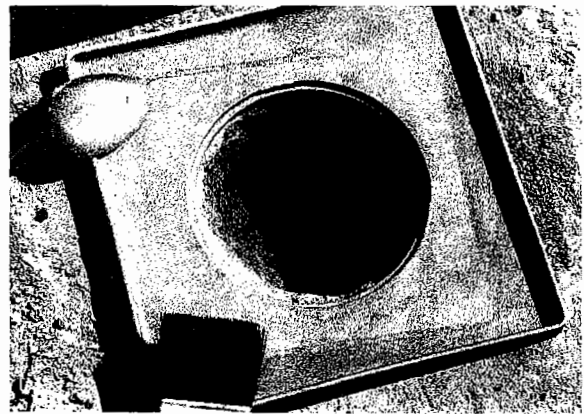
Field density test using sand cone.



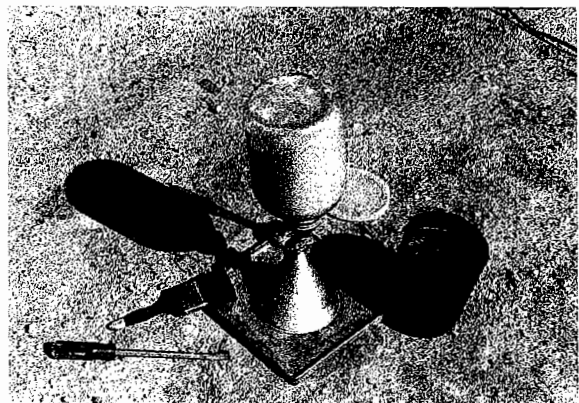
(a) Digging tools, sample can (coffee can painted gray) and 60-mm paint brush to clean template. Carefully smooth site and place template as shown. Backfill any holes caused by removal of surface rocks.



(b) Digging hole. Screwdriver can be used to loosen soil which is then spooned into can as shown. Carefully remove and save larger stones as shown in can. If stones are much larger than 35 or 40 mm, the site may be unsuitable for this size of sampling equipment.



(c) Hole completed. Hole should be carefully cleaned of soil crumbs and be on the order of 100 to 150 mm deep (producing a can three-fourths to nine-tenths full).



(d) Carefully place sand jug onto template and open flow valve. Avoid local vibrations as much as possible. Note can three-fourths full of soil (shaken down). Can is left open to show amount—normally can is covered as soon as sampling is complete to avoid loss of moisture.

The following procedure can be used to determine the density of the sand used in the sand-cone apparatus:

1. Weigh a standard (944 or 1000 cm³) compaction mold with base plate attached. Strictly, the volume should be determined also, but it can be assumed to have the correct volume within laboratory accuracy. Note that any available container of known volume of reasonably regular shape can be used in lieu of the compaction mold.
2. Carefully pour sand into the mold from the same approximate height as the sand falls into the hole in the field. Strike off the top of the mold with a steel straightedge and weigh.
3. Repeat step 2 until you obtain two readings in good agreement (say, within about 10 g), average the results of these two readings, and put on the blackboard.
4. Average the values obtained by all the groups as displayed on the blackboard, except that any group whose value is substantially out of agreement with the other groups will have to do this part of the project over. The *average value* of density is to be recorded on the data sheet in the appropriate location and used by all groups for hole-volume determinations.

To determine the weight of sand to fill the sand cone and template groove:

1. Place the template on a flat surface. Weigh the sand cone with attached jug full of sand and record the weight.
2. Turn the sand cone upside down with the valve closed and place on the template. Open the valve and let the sand pour until it stops; then close the valve.
3. Reweigh the sand jug and the remaining sand. The difference between weights obtained in steps 1 and 2 is the weight of sand to fill the cone and template groove. Repeat this series of steps for a second weight value. Average the two values and put on the blackboard.
4. Average the values of sand-cone weights (*if all the groups have the same type of sand cone*) for a value to use on the data sheet.

To calibrate the balloon apparatus, the following procedure is recommended:

1. Place the template on a flat surface and obtain a zero reading by pumping the water against the flat surface.
2. Next, place the template over a standard compaction mold or other container of known volume that will not rupture the rubber balloon.
3. Place the balloon apparatus on the template and pump the balloon into the mold at least three times. Take the average of the three final readings (which should be very nearly identical). Try to use the same balloon pressure as used in the field (either by gage or by estimation).
4. The difference between the zero and final averaged readings is the measured volume of the container ΔR .

$$\Delta R = \text{final reading} - \text{zero reading}$$

5. The correction factor CF is computed by proportion as

$$\Delta R(CF) = \text{known volume}$$

from which the correction factor is

$$CF = \frac{\text{known volume}}{\Delta R} \quad (10-4)$$

All groups should show their correction factor on the blackboard so that a class average value can be obtained. If the CF is too small to affect the field density beyond 0.01, neglect it.

The Report

The discussion should comment on possible limitations of the procedure. In the "Conclusion" show a tabulation of sand density and the weight of sand to fill the cone. List the balloon density CF . Tabulate the wet and dry densities obtained in the field along with water-content data. Compare the two methods of determining field density. In the "Discussion" answer the following two questions:

1. What material(s) other than sand can be used to find the volume of the hole?
2. What problem may develop in using the balloon-density apparatus in loose or low-plasticity soils?

Experiment No. 11

COEFFICIENT OF PERMEABILITY—CONSTANT HEAD METHOD

References

AASHTO T215-66

ASTM D2434-68

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Johnson, A. I., and R. C. Richter (1967), Selected Bibliography on Permeability and Capillarity Testing of Soil and Rock Materials, *ASTM STP no. 417*, pp. 176-210.

Mitchell, J. K., D. R. Hooper, and R. G. Campanella (1965), Permeability of Compacted Clay, *J. Soil Mech. Found. Div., ASCE, SM 4*, July, pp. 41-65.

Mitchell, J. K., and J. S. Younger, (1967), Abnormalities in Hydraulic Flow Through Fine-Grained Soils, *ASTM STP no. 417*, pp. 106-141.

Terzaghi, K., (1925), Principles of Soil Mechanics—III, *Eng. News-Record*, vol. 95, p. 832.

Objective¹

To introduce the student to one method of determining the coefficient of permeability of a granular soil.

Equipment

Permeability device (Fig. 11-1 or Fig. 12-1)

Timer

Thermometer

Graduated cylinder

General Discussion

The coefficient of permeability is a constant of proportionality relating to the ease with which a fluid passes through a porous medium. Two general laboratory methods are available for determining the coefficient of permeability of a soil directly. These are the *constant-head method*, here described, and the *falling-head method* of Experiment No. 12. Both methods uses Darcy's law

$$v = ki$$

and the corresponding flow rate is

$$q = kiA$$

where

q = quantity of fluid flow in a unit time

k = coefficient of permeability (units of velocity)

i = hydraulic gradient = h/L

h = differential head across the sample

L = sample length across which h is measured

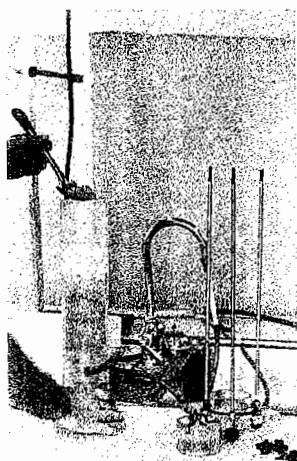
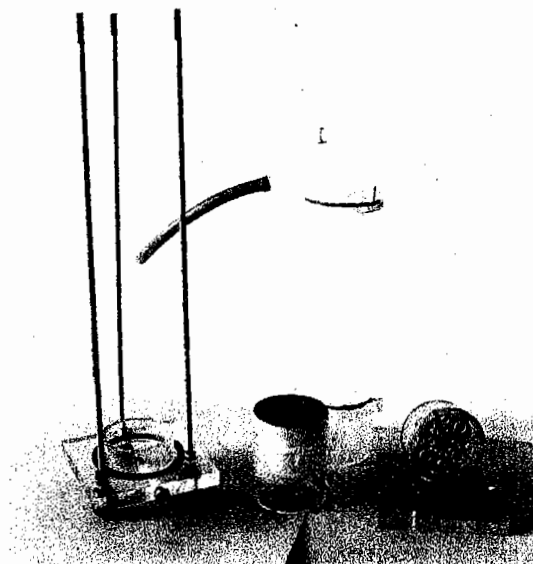
A = cross-section area of soil mass under consideration

¹The author requires Experiments No. 11 and 12 be done during the same laboratory period on the same soil sample.

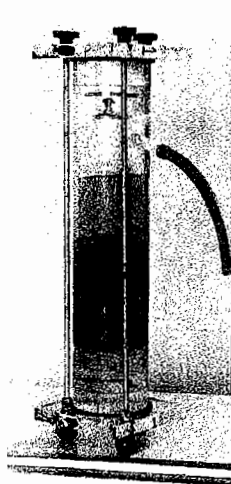
Figure 11-1

A permeability device for granular materials designed by author.

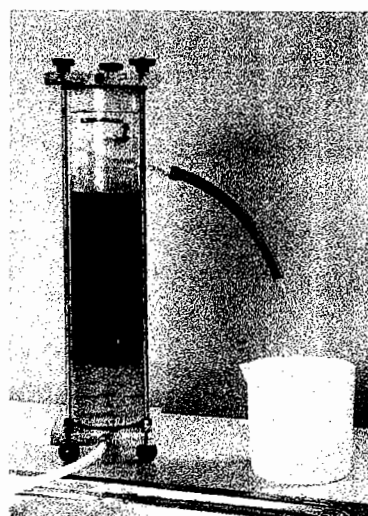
(a) Device disassembled; two end pieces clamped together with threaded rods; piece in foreground contains top of sample with No. 200 mesh screen and also the overflow weir shown in Fig. 11-1c; base portion includes a No. 200 mesh screen to contain base of sample and also the diffuser shown which spreads water over sample base.



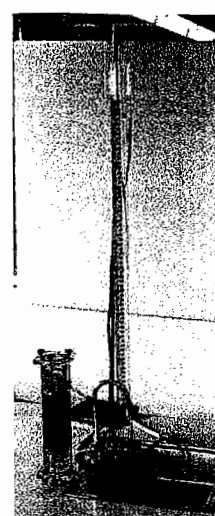
(b) Building sample—dry sand is shown, but wet sand can be used. With care sample can be built to reasonably large density.



(c) Sample being saturated with upward flow of water. Note there is no way sample volume can change when device is assembled, regardless of hydraulic gradient.



(d) Water flowing through sample, over top of overflow weir, and into outlet tube as test is run.



(e) Constant-head setup. Note overflow container at top of standpipe to control head in standpipe.

Neither the constant-head nor the falling-head laboratory method provides a reliable value for the coefficient of permeability of a soil. Reasons for this are varied, but the major ones are as follows:

1. The soil in the permeability device is never in the same state as in the field—it is always disturbed to some extent.

2. Orientation of the in situ stratum to the flow of water is probably not duplicated. In sands, the ratio of horizontal flow to vertical flow may be 3 to 4 or more ($k_h/k_v = 3$ or more), and this generally always occurs. Even if the field void ratio is duplicated for sands, the relationship of k_h/k_v will probably be lost. Clayey fills will have horizontal jointing due to placing the fill in lifts of 15 to 30 cm and rolling. This results in $k_h \neq k_v$, again a situation hard to duplicate in the laboratory.
3. Boundary conditions are not the same in the laboratory. The smooth walls of the permeability mold make for better flow paths than if they were rough. If the soil is stratified vertically, the flow in the different strata will be different, and this boundary condition may be impossible to duplicate.
4. The hydraulic head h may be different (often much larger) in the laboratory, causing a washout of fine material to the boundary, with a possible reduction of k . The field hydraulic gradient ($i = h/L$) is on the order of 0.5 to 1.5, whereas in the laboratory it may be 5 or more. Some evidence indicates that $v = ki$ is not linear for all values of i , especially the larger values [Mitchell and Younger (1967)]. On the other hand, there is evidence that for fine-grained soils (clays), there may be some *threshold gradient* below which no flow will take place [Terzaghi (1925)].
5. The effect of entrapped air on the laboratory sample will be large even for small air bubbles since the sample is small.

The *coefficient of permeability* of a homogeneous, isotropic soil mass depends primarily on the following factors:

1. The viscosity of the pore fluid (usually water). As the temperature increases, the viscosity of water decreases and the coefficient of permeability increases; i.e., the flow rate increases. The coefficient of permeability is standardized at 20°C, and the coefficient of permeability at any temperature T is related to k_{20} as in the following equation:

$$k_{20} = k_T \frac{\eta_T}{\eta_{20}} \quad (11-1)$$

where η_T and η_{20} are the viscosities of the fluid at the temperature T of the test and at 20°C, respectively. Either absolute or kinematic fluid viscosity may be used in Eq. (11-1). Use Table 12-1.

2. The void ratio e of the soil. Various attempts have been made to relate the coefficient of permeability of the soil at one void ratio to the same soil at a different void ratio [$k = f(e)$] by expressions such as

$$k_2 = k_1 \left(\frac{e_2}{e_1} \right)^2 \quad (11-2)$$

$$k_2 = k_1 \frac{e_2^3/(1 + e_2)}{e_1^3/(1 + e_1)} \quad (11-3)$$

For clays [Terzaghi (1925)], since $k \approx 0$ at the void ratio of 0.15,

$$k_2 = k_1 \frac{\eta_1 (e - 0.15)^{11} (1 + e)}{\eta_2 (e - 0.15)^8 + 0.0166} \quad (11-4)$$

Equation (11-4) should be used with caution; it is recommended that one perform permeability tests at various void ratios to obtain the value of k rather than use Eq. (11-4). In Eq. (11-4), k_1 is the coefficient of permeability at $e = 1.0$.

3. The size and shape of the soil grains. Allan Hazen studied filter sands for use in water works (ca. 1890) and concluded that for *clean sands* and *gravels* the coefficient of permeability could be expressed approximately as

$$k = 100D_{10}^2 \quad \text{cm/s} \quad (11-5)$$

for a range of D of $0.1 < D_{10} < 3.0$ mm. The D_{10} used here is the percent finer size from the grain-size distribution curve (as Experiment No. 5) and in centimeters.

Angular and platy particles tend to reduce k more than when the soil predominates in rounded and more spherical soil particles.

4. The degree of saturation. As the degree of saturation increases, the apparent coefficient of permeability also increases. Part of this increase is due to the breaking of surface tension. The remainder is an unknown quantity since it is difficult to determine k unless one considers the continuity of flow through the medium. The flow through the medium can only be measured by considering the quantity going into and coming out of the soil mass. As an extreme case one could, in a dry soil, have a considerable flow into the sample and no flow at all out. A computation of k would yield $k = 0$, which is obviously incorrect. Saturated samples are generally used in the laboratory to avoid this problem, although a few relatively recent investigations have been concerned with soil specimens where $S < 100$ percent [Mitchell et al. (1967)].

A widely used permeameter is the *standard compaction permeameter*, which uses a 944-cm³ standard compaction mold for determining the permeability of compacted specimens, as for dam cores, levees, etc. This does not preclude the use of other permeability devices, such as those shown in Fig. 11-1 (of plastic tubing), which can be fabricated directly in the laboratory and are relatively inexpensive.

In the laboratory methods, the full hydraulic drop (or head loss) is assumed to occur across the soil sample, whereas a small amount of the hydraulic drop will occur across the porous stone in the base of the permeability device. This can be avoided by constructing the device as shown in Fig. 11-1, so that the head loss across the two 200-mesh screens at the sample ends is negligible.

Laboratory tests to determine the coefficient of permeability often require the use of deaired (and sometimes distilled) water. The use of deaired water may have some merit since a bubble of air coming out of solution in the small laboratory sample may be very important compared with the effect of the same-size bubble in the field soil. In the author's opinion, except for precise research, the use of deaired water introduces questionable increase in precision of k determinations for routine laboratory work when considering the inaccuracies of this test and that the in situ water is neither deaired nor distilled. Two fairly simple procedures may be used to reduce the dissolved-air problem. One technique is to use water for the test which is warmer than the soil sample so that the water cools as it percolates through the soil; this will attract air from the sample into solution. The other technique is to use a large enough gradient i so that the pressure holds or forces the free air into solution.

For permeability tests on soils of low permeability (say, 10^{-4} to 10^{-9} cm/s), it is more practical to use a thin sample about 1 in. (2 to 3 cm) thick in a special permeameter [Mitchell and Younger (1967)]. Alternatively, the coefficient of permeability can be computed from the consolidation test data (Experiment No. 13) using c_r from Eq. (13-8) as

$$k = \frac{\alpha_r \gamma_w c_v}{1 + e} \quad (11-6)$$

with terms identified in Experiment No. 13. These latter procedures are beyond the scope of the experiments to be performed in this text.

Permeability tests on soils of low permeability must be performed very carefully for the results to have meaning. Any leakage from the permeability device, or evaporation from the standpipe or from the collection container, and the flow along the less-resistant flow paths along the interface of the permeability device and soil, will materially affect the computed value of k . As an example let us assume a constant-head test, a soil sample with $A = 81 \text{ cm}^2$ (approximate area of standard compaction mold), a hydraulic gradient of 20 (which is very large even for laboratory testing), and a real coefficient of permeability of $1 \times 10^{-6} \text{ cm/min}$. How much water Q will be collected in 1 h for determination of k ?

Since $Q = Ait$

$$Q = (1 \times 10^{-6} \text{ cm/min})(20)(81)(60 \text{ min/h}) = 0.0972 \text{ cm}^3/\text{h}$$

This is a negligible quantity and could easily evaporate as fast as it is collected unless special precautions are taken. If water travels along the wall of the permeability device at a rate of $1 \times 10^{-4} \text{ cm/min}$, twenty times as much water will be collected. A hydraulic gradient of 20 is far in excess of any likely in situ value and this, too, could invalidate the results.

Procedure *This Will Be a Group Project*

1. Weigh the container of material (granular) to be used for the test.
2. Assemble the permeability device as necessary to place soil in it for the test. Determine the volume if not already known.
3. Each group should make a test sample to a different density than any other group. This may be done by placing the soil loosely, placing it with some vibration, or placing it with considerable effort. Write the density of the sample on the blackboard so that no two groups have the same density. Try to vary the density by as much as 0.4 to 0.6 kN/m^3 between groups.

For the compaction permeameter do steps 4 through 6 in order

4. Place a piece of filter paper on top of the sand, carefully clean the rim of the mold, place a rubber gasket on the rim, and then firmly seat the cover. The cover should have a transparent piece of plastic tubing to which the water inlet will be later attached. Attach a length of rubber tubing to the outlet pipe.
5. Place the permeameter in a sink in which the water is about 5 cm above the cover. Be sure the outlet pipe is open so that water can back up through the sample to produce saturation with a minimum of entrapped air. When water in the plastic inlet tube on top of the mold reaches equilibrium with the water in the sink, the sample may be assumed to be saturated. A soaking period of 24 h might provide better results, but for the objectives of this experiment involves too much time. Note, however, that this procedure may expand the sample somewhat inside the mold.
6. With the water level stabilized in the inlet tube, take a hose clamp and clamp the exit hose. Remove the permeameter from the sink and attach the inlet to the constant-head standpipe.
7. Deair the lines at the top of the soil sample by opening the hose clamp on the inlet and opening the petcock on top of the mold cover (refer to Fig. 12-1). When no more air comes out, close the petcock. Measure the hydraulic head across the sample.

For using the Bowles type of permeability device (Fig. 11-1 and 11-2)

4. (a) Complete the assembly of the device. Orient the inlet and exit tubes conveniently for collection of water and initial saturation/drainage.

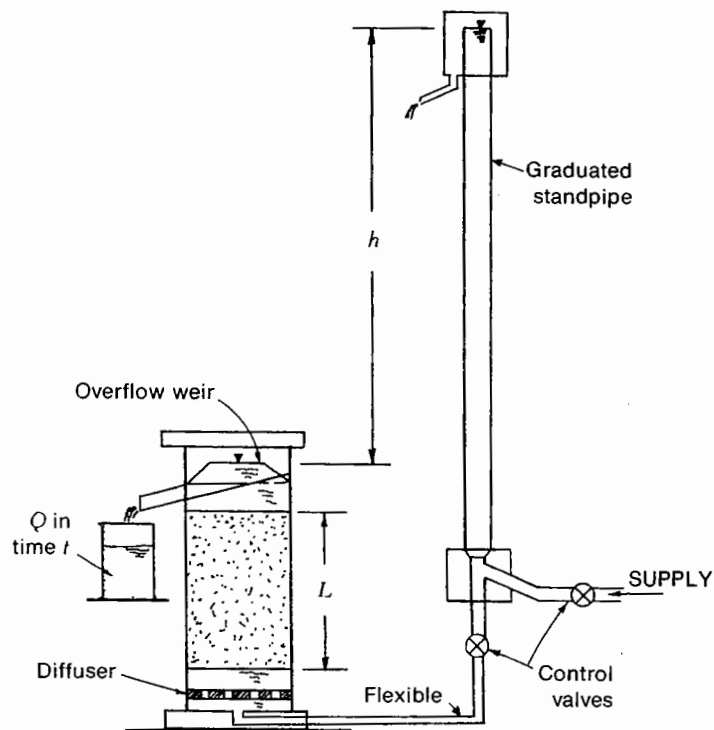


Figure 11-2
Line details of constant-head test as illustrated in Fig. 11-1.

5. (a) Connect the water inlet to the water supply. Next slowly allow the sample to saturate (visual observation), and flow conditions to stabilize by allowing some flow over the overflow weir and out. Clamp the inlet hose and go to step 8.
8. Using a 500 or 1000 ml container (larger is preferable), record the time to collect 750 to 900 ml of water. Take the water temperature and repeat 2 to 3 additional times using the same time ($t = \text{constant}$). The amount of water collected on successive runs usually decreases and should be considered in your "Discussion."

Record all the data from the experiment on the data sheet for this experiment.

9. Each group should compute its k value for the temperature of the test. Each group should also compute k_{20} for your test (with $t = \text{constant}$, T should be constant if set up is stabilized and you can average Q and compute a single value for k_T ; if these conditions are not satisfied, compute the individual k_T values and then the separate k_{20} values). Place the average k_{20} value on the blackboard alongside your density value. Obtain viscosity corrections for temperature from Table 12-1.

The coefficient of permeability is computed as

$$k = \frac{QL}{Aht} \quad \text{cm/s}$$

Be sure the computations are checked and that units are reported consistently (as cm/s) for all groups.

10. Using a value of G_s furnished by the instructor (or estimated by you), compute the void ratio for each density and plot a curve of k_{20} vs. e . Draw a smooth curve through the points (reject any point which does not reasonably follow a trend).
11. Find an equation which fits the plotted curve reasonably well. Plot this equation on the curve from step 10, using a colored pencil (not red) or with a dashed

COEFFICIENT OF PERMEABILITY (Constant Head, ~~Falling Head~~)

Data Sheet 13

Project K For Lab Manual Job No. —
 Location of Project Bradley University (using permeameter of Fig. 11-1)
 Description of Soil Br. Med. Coarse Sand w/trace Gravel
 Tested by JEB Date of Testing 6-7-76
 Sample Dimensions: Diam. 7.62 cm; Area 45.6 cm²; Ht. 20.32 cm
 Wt. soil + pan Init. 2984.2 g Vol. 926.7 cm³
 Wt. soil + pan Final 1427.3 g Unit wt. 16.50 kN/m³
 Wt. of Sample 1556.9 g

Constant Head

$h = 87.0$ cm

Test data

Test data used

Test No.	t, s	Q, cm ³	T, °C	Test No.	t, s	Q, cm ³	T, °C
1	180	775	23		180	775	
2	"	772	22		"	772	
3	"	761	22		"	761	
4							
Average "					180	769.3	22

$$k_T = QL/Aht = \frac{769.3(20.32)}{45.6(87)(180)}$$

$$\eta_T/\eta_{20} = 0.9531$$

$$= 0.022 \text{ cm/s}$$

$$k_{20} = k_T \eta_T/\eta_{20} = 0.021 \text{ cm/s}$$

Falling Head

Standpipe = [burette, other (specify)] _____

Area of standpipe, $a =$ _____ cm²

Test data^b

Test data used

Test no.	h_1 , cm	h_2 , cm	t, s	Q_{in} , cm ³	Q_{out} , cm ³	T, °C	Test no.	h_1 , cm	h_2 , cm	t, s	T, °C
1											
2											
3											
4											
Average											

$$\eta_T/\eta_{20} =$$

$$k_T = \frac{aL}{At} \ln \frac{h_1}{h_2} = \text{_____ cm/s}$$

$$k_{20} = k_T \eta_T/\eta_{20} = \text{_____ cm/s}$$

^aUse averaged values only if there is a small difference in test temperature, say, 1–2°C.

^bThis test can be considerably simplified by using the same values of h_1 and h_2 each time, otherwise you cannot average these values regardless of T.

Figure 11-3

Data from a constant-head permeability test using equipment in Fig. 11-1.

line. Also plot a curve of

$$k = k_1 \left(\frac{e_2}{e_1} \right)^2$$

on the graph using a different-color pencil as a second comparison. Limit the range of e values to between 0.4 and 1.1.

12. Compute the approximate actual water velocity as

$$v_a \approx \frac{1+e}{e} v \quad (\text{cm/s})$$

for your test.

13. In your report:

- a. Compare equations and test data curve of step 11. How does the approximate equation given in step 11 compare, in particular?
- b. Do you think the actual velocity produces turbulent flow?
- c. Is the collection of a lesser quantity of water in the same time due to formation of a filter skin or to air coming out of solution? What can be done to produce a flow of $Q = \text{constant}$?
- d. Discuss any changes which you think could be made to improve this test as a practical possibility.

Experiment No. 12

COEFFICIENT OF PERMEABILITY – FALLING-HEAD METHOD

References

See Experiment No. 11

Objective

To introduce the student to a method of determining the coefficient of permeability of a fine-grained soil (such as fine sand, silt, or clay). The test may also be used for coarse-grained soils.

Equipment

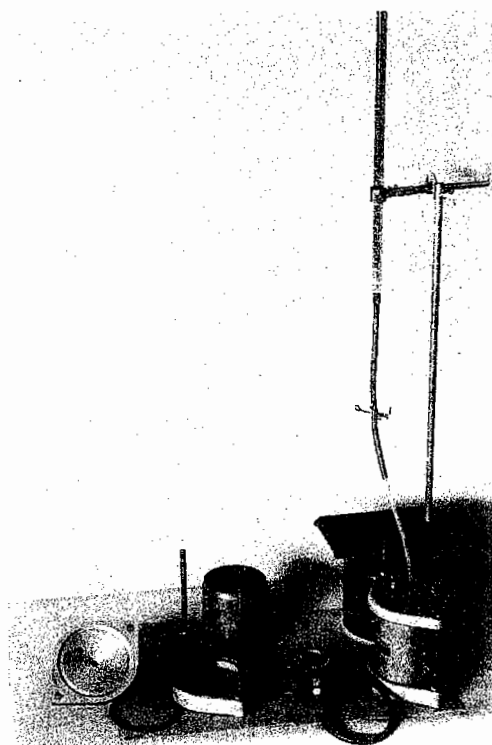
Permeability device
Timer
Thermometer
Ring stand with test-tube clamp or other means to develop a differential head across soil sample.
Burette (to use with ring stand or other means of support)

General Discussion

The general discussion of Experiment No. 11 is also applicable to this experiment. The limitations of the constant-head test are inherent in this test, and in addition, tests of long duration will require some way of controlling evaporation of water in the standpipe (Fig. 12-1).

Figure 12-1

Falling-head permeability test using the standard compaction-mold permeameter. Shown is both a disassembled device and a test setup using a 100-ml burette. Note the very thick porous stone in the base and the small opening in the cap assembly which may erode a depression into the sample with a large hydraulic gradient. Great care is required in assembly to avoid leaks. Use a meter stick to measure the hydraulic heads, h_1 , and h_2 . With small hydraulic gradients exercise care that sample does not drain.



The equation applicable to this experiment can be derived (making reference to Fig. 12-2), but is left as part of the exercise for the student report and will be merely presented:

$$k = \frac{aL}{At} \ln \frac{h_1}{h_2} \quad (12-1)$$

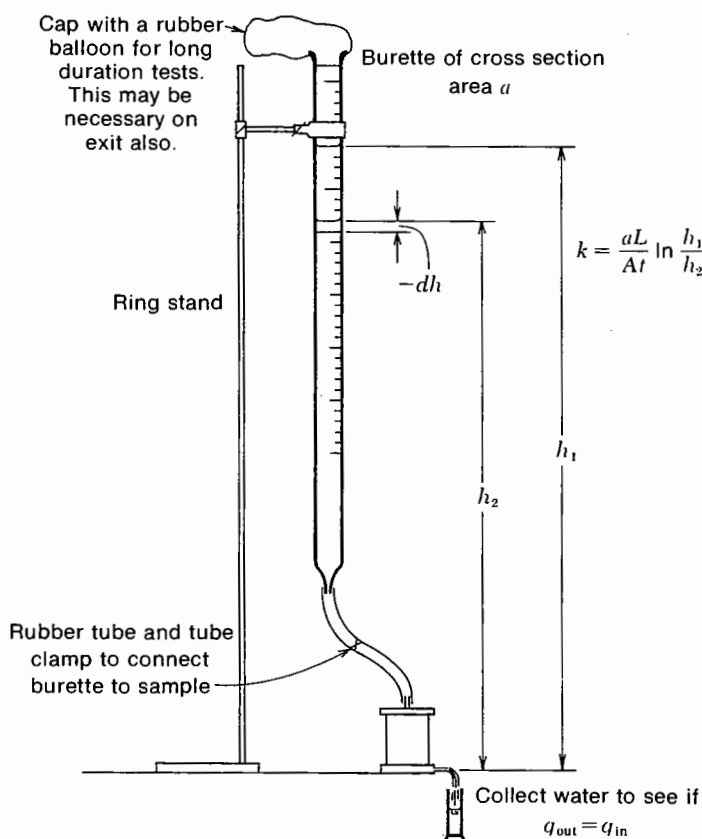


Figure 12-2
Line details of the falling-head test as illustrated in Fig. 12-1.

where a = cross-section area of burette or standpipe (Fig. 12-2), cm^2
 A = cross-section area of soil sample, cm^2
 h_1 = hydraulic head across sample at beginning of test ($t = 0$)
 h_2 = hydraulic head across sample at end of test ($t = t_{\text{test}}$)
 L = length of soil sample, cm
 t = elapsed time of test, s
 \ln = logarithm to base 2.7182818...

It should be noted that this method of determining the coefficient of permeability k is primarily for economy since a test to determine k for a fine-grained soil may be of several days' duration. The constant-head test of Experiment No. 11 would consume a rather large quantity of water in the laboratory to maintain a constant head for most setups. For long-duration tests and where the quantity of flow through the soil sample is small, one should make some provision to control evaporation of the water in the standpipe and to avoid sample drainage and/or evaporation from the exit tube from the sample. One solution for this problem is to do the test in a controlled-humidity room. Another is to keep the standpipe reser-

voir covered with a small rubber balloon which has been partially inflated. (Do not plug the standpipe as a vacuum will eventually form, stopping the flow of water.) To control sample drainage and ensure that the exit tube flows full (or to control tail-water evaporation), submerge the exit tube in a container of water. Obtain the tail-water elevation for h_2 according to the various laboratory setups. Use judgment and ingenuity to control any sample leaks.

Procedure *This Is a Group Project¹*

1. Build the soil sample using the instructions given in Experiment No. 11.
2. Fill the burette (or other standpipe) to a convenient height and measure the hydraulic head across sample to obtain h_1 .
3. Commence the flow of water and simultaneously start timing the test. Allow water to flow through the sample until the burette (or standpipe) is almost empty or to a convenient mark. Simultaneously stop the flow and timing. Obtain the head h_2 . Take the temperature of the test.

If it is necessary to obtain the area of the standpipe a , collect the water in a beaker.

4. Refill the burette (or standpipe) and repeat the test two additional times. Use the same h_1 and h_2 values and obtain the corresponding elapsed times. Take the temperature for each run.

If it is necessary to compute the area of standpipe, collect the water for each test run and *accumulate* in a graduated cylinder. After the last test run obtain the average Q_{test} as

$$Q_{\text{test}} = \frac{Q_{\text{total}}}{\text{no. of tests}}$$

from which a is accurately and readily computed. This computation is not necessary if a graduated burette is used.

If this test is done with Experiment No. 11 and the constant-head data has not been obtained, take the data for Experiment No. 11 next.¹

5. Each individual should compute the coefficient of permeability at the test temperature, k_T and for 20°C. Obtain viscosity corrections from Table 12-1. Use a data sheet from the data sheet section. Average the results for k (note that a single value can be computed if the temperature does not vary more than 1 or 2°C and you used h_1 and $h_2 = \text{constant}$ for all runs since time can be averaged under these conditions).

Refer to Fig. 12-3 for a typical set of falling-head data.

6. In your report
 - a. Discuss test limitations (specifically for your test setup)
 - b. Can you propose a better (and practical) way of doing the test?
 - c. Compare the k values between Experiments No. 11 and No. 12 (if both are done on the same sample). What could cause any differences in the two values?
 - d. Show the derivation of Eq. (12-1) in sample computations.
 - e. How long will it take for h_2 to be zero?

¹The author does Experiment No. 11 first, but either test can be done first.

Table 12-1 Viscosity Corrections for $\eta T/\eta_{20}$

°C	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
10	1.3012	1.2976	1.2940	1.2903	1.2867	1.2831	1.2795	1.2759	1.2722	1.2686
11	1.2650	1.2615	1.2580	1.2545	1.2510	1.2476	1.2441	1.2406	1.2371	1.2336
12	1.2301	1.2268	1.2234	1.2201	1.2168	1.2135	1.2101	1.2068	1.2035	1.2001
13	1.1968	1.1936	1.1905	1.1873	1.1841	1.1810	1.1777	1.1746	1.1714	1.1683
14	1.1651	1.1621	1.1590	1.1560	1.1529	1.1499	1.1469	1.1438	1.1408	1.1377
15	1.1347	1.1318	1.1289	1.1260	1.1231	1.1202	1.1172	1.1143	1.1114	1.1085
16	1.1056	1.1028	1.0999	1.0971	1.0943	1.0915	1.0887	1.0859	1.0803	1.0802
17	1.0774	1.0747	1.0720	1.0693	1.0667	1.0640	1.0613	1.0586	1.0560	1.0533
18	1.0507	1.0480	1.0454	1.0429	1.0403	1.0377	1.0351	1.0325	1.0300	1.0274
19	1.0248	1.0223	1.0198	1.0174	1.0149	1.0124	1.0099	1.0074	1.0050	1.0025
20	1.0000	0.9976	0.9952	0.9928	0.9904	0.9881	0.9857	0.9833	0.9809	0.9785
21	0.9761	0.9738	0.9715	0.9692	0.9669	0.9646	0.9623	0.9600	0.9577	0.9554
22	0.9531	0.9509	0.9487	0.9465	0.9443	0.9421	0.9399	0.9377	0.9355	0.9333
23	0.9311	0.9290	0.9268	0.9247	0.9225	0.9204	0.9183	0.9161	0.9140	0.9118
24	0.9097	0.9077	0.9056	0.9036	0.9015	0.8995	0.8975	0.8954	0.8934	0.8913
25	0.8893	0.8873	0.8853	0.8833	0.8813	0.8794	0.8774	0.8754	0.8734	0.8714
26	0.8694	0.8675	0.8656	0.8636	0.8617	0.8598	0.8579	0.8560	0.8540	0.8521
27	0.8502	0.8484	0.8465	0.8447	0.8428	0.8410	0.8392	0.8373	0.8355	0.8336
28	0.8318	0.8300	0.8282	0.8264	0.8246	0.8229	0.8211	0.8193	0.8175	0.8157
29	0.8139	0.8122	0.8105	0.8087	0.8070	0.8053	0.8036	0.8019	0.8001	0.7984
30	0.7967	0.7950	0.7934	0.7917	0.7901	0.7884	0.7867	0.7851	0.7834	0.7818
31	0.7801	0.7785	0.7769	0.7753	0.7737	0.7721	0.7705	0.7689	0.7673	0.7657
32	0.7641	0.7626	0.7610	0.7595	0.7579	0.7564	0.7548	0.7533	0.7517	0.7502
33	0.7486	0.7471	0.7456	0.7440	0.7425	0.7410	0.7395	0.7380	0.7364	0.7349
34	0.7334	0.7320	0.7305	0.7291	0.7276	0.7262	0.7247	0.7233	0.7218	0.7204
35	0.7189	0.7175	0.7161	0.7147	0.7133	0.7120	0.7106	0.7092	0.7078	0.7064

COEFFICIENT OF PERMEABILITY (Constant Head, Falling Head)

Data Sheet 13

Project Falling Head Test Job No.
 Location of Project Bradley University (std. Comp. mold)
 Description of Soil Lt. Br. U. Fine sand
 Tested by JEB Date of Testing 7-9-76
 Sample Dimensions: Diam. 10.2 cm; Area 81.1 cm²; Ht. 11.6 cm
 Wt. soil + pan Init. 3074.9 g Vol. 944.0 cm³
 Wt. soil + pan Final 1525.1 g Unit wt. 16.10 kN/m³
 Wt. of Sample 1549.8 g

Constant Head

$h =$ _____ cm

Test data

Test data used

Test No.	t, s	Q, cm ³	T, °C	Test No.	t, s	Q, cm ³	T, °C
1							
2							
3							
4							
Average "							

$$k_T = QL/Aht =$$

$$\eta_T/\eta_{20} =$$

$$= \text{_____ cm/s}$$

$$k_{20} = k_T \eta_T/\eta_{20} = \text{_____ cm/s}$$

Falling Head

Standpipe = [burette, other (specify)] 100 ml. burette

Area of standpipe, $a =$ 1.71 cm²

Test data^b

Test data used

Test no.	h_1 , cm	h_2 , cm	t, s	Q_{in} , cm ³	Q_{out} , cm ³	T, °C	Test no.	h_1 , cm	h_2 , cm	t, s	T, °C
1	51.1	24.3	54.1		45.8	21					
2	51.1	24.3	54.7		"	"					
3	51.1	24.3	55.3		"	"					
4											
Average								51.1	24.3	54.7	21

$$a = \frac{45.8}{51.1 - 24.3} = 1.71 \text{ cm}^2$$

$$\eta_T/\eta_{20} = 0.9761$$

$$k_T = \frac{aL}{At} \ln \frac{h_1}{h_2} = \frac{1.71(11.6)}{81.1(54.7)} \ln \frac{51.1}{24.3} = 3.32 \times 10^{-3} \text{ cm/s}$$

$$k_{20} = k_T \eta_T/\eta_{20} = 3.24 \times 10^{-3} \text{ cm/s}$$

^aUse averaged values only if there is a small difference in test temperature, say, 1-2°C.

^bThis test can be considerably simplified by using the same values of h_1 and h_2 each time, otherwise you cannot average these values regardless of T.

Figure 12-3

Data from a falling-head permeability test using the standard compaction mold and general details of Fig. 12-1.

Experiment No. 13

CONSOLIDATION TEST

References

- AASHTO T216-66
ASTM D2435-70
AASHTO(1976), Estimation of Consolidation Settlement, *Transportation Research Board, Special Report no. 163* (with several references).
Casagrande, A. (1936), The Determination of the Pre-Consolidation Load and its Practical Significance, *Proc. 1st Int. Conf. Soil Mech. Found. Eng., Harvard*, vol. 3, pp. 60-64.
Crawford, C. B. (1964), Interpretation of the Consolidation Test, *J. Soil Mech. Found. Div., ASCE, SM 5*, September, pp. 87-102.
Leonards, G. W., and A. G. Altschaeffl (1964), Compressibility of Clay, *J. Soil Mech. Found. Div., ASCE, SM 5*, September, pp. 133-156.
Lo, K. Y. (1961), Secondary Compression of Clays, *J. Soil Mech. Found. Div., ASCE, SM 4*, August, pp. 61-87.

Equipment

- Consolidometer¹ (refer to Fig. 13-1)
- Dial gage² reading to 0.01 mm (or 0.0001 in)
- Loading device
- Stop watch or timer
- Sample trimming equipment as available or necessary.

General Discussion

When any soil is subjected to an increase in pressure (or load), a readjustment in the soil structure occurs which may be considered as consisting primarily of *plastic deformation* with a corresponding *reduction in void ratio*. A small amount of *elastic deformation* may also take place, but considering the magnitude of loads (or contact pressures) involved and that the modulus of elasticity of the soil grains is on the order of 20 MPa, the elastic deformation (which is recoverable if the load is removed) is negligible.

When a load is applied to a coarse-grained soil which is either dry, partially saturated, or fully saturated, or when a load is applied to a dry fine-grained soil, the process of plastic deformation with void-ratio reduction takes place in a short enough period of time for the process to be considered instantaneous. This may be explained for dry soils by the fact that air has an extremely low viscosity and is easily compressed; thus the soil solids offer no resistance to the outward flow of the pore fluid (air, in this case) as the soil voids are reduced. In the case of a satu-

¹The standard 2½-in consolidometer can be converted to SI [to use the loading device (¼ ton per square foot (t/ft²) = 25 kilopascals (kPa); ½ t/ft² = 50 kPa; 1 t/ft² = 100 kPa, etc.] as follows: (a) Purchase a piece of brass tubing of appropriate OD and ID such that a machinist can turn a new ID = 62.14 mm (vs. 63.5 mm of standard 2½-in ring). Make the height 24 mm. (b) Turn a piece of brass or aluminum to make a trim disk using 2 mm and 4 mm (refer to Fig. 13-1) to produce a final sample thickness of 20 mm. (c) Use a grinding wheel and carefully grind down the necessary porous stones. (d) Machine a piece of brass for a new load head (or turn down existing head).

²Dial gages normally used with this test recorded to 0.0001 in which was far too much precision considering how the sample thickness is obtained. The 0.01-mm gage is a precision of 0.0004 in which is certainly precise enough.

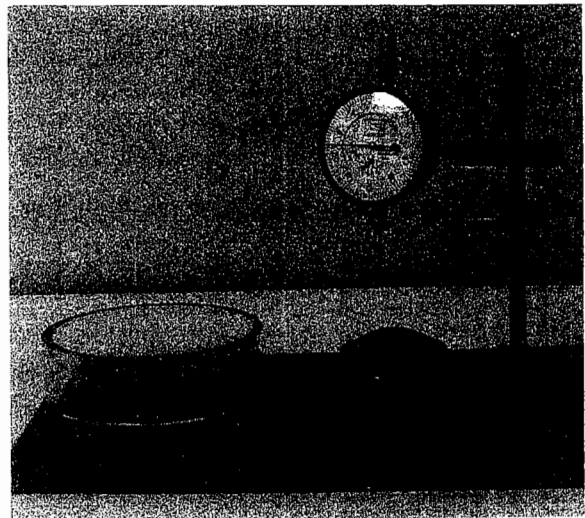
Figure 13-1

Floating-ring consolidometer test and equipment.

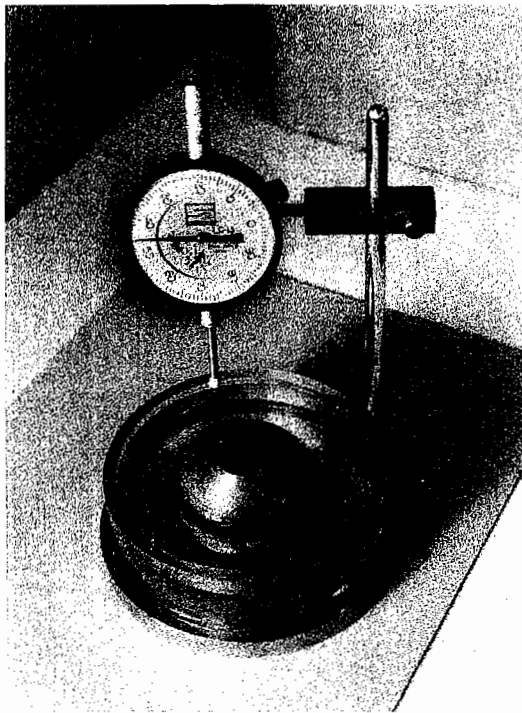
(a) Consolidometer equipment: Brass ring (62.14×24 mm), sample cutter to cut sample of required diameter to extrude into brass consolidometer ring; two porous stones, brass load block and a trimming disk with a 2- and 4-mm trim depth (4-mm depth turned up). Sample is fitted top and bottom with porous stones and fitted into consolidometer base which has been recessed to provide correct alignment. Plastic ring which surrounds sample and fits into base groove (grease with silicone grease) is used as a saturation device (add water as evaporation occurs so water level is always above upper porous stone). A fixed-ring consolidometer is very similar, and if consolidometer weight is same (add weight or use metal drill to remove metal, or as required), the same loading system can be used for either.



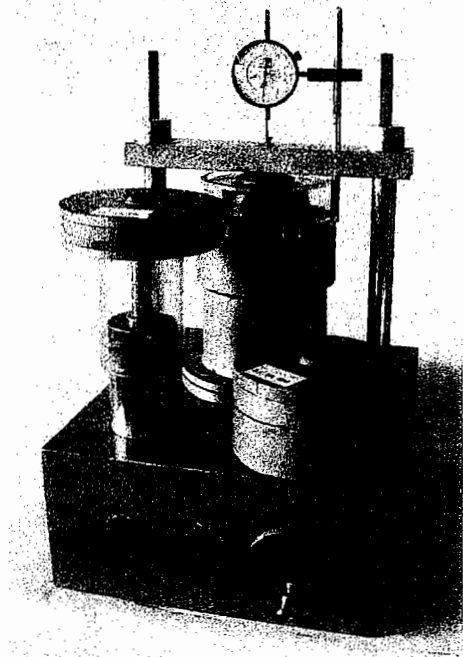
(b) After soil sample is placed in the brass ring, use trimming disk to extrude 2 mm of sample, trim this flush, and place on a filter paper to avoid sticking. Then pull trim disk turn over, use 4-mm side to push sample back through brass ring, trim opposite side flush (produces accurate sample thickness), and place on second filter paper. Now remove trim disk, add porous stones, and center sample in brass ring.



(c) Place sample in consolidometer base and add brass load block.



(d) Add plastic ring after carefully greasing groove and partially fill with water and check for leaks. If rim leaks, remove and regrease. If no leaks are found, place in load device and commence test.



(e) Test in progress using a loading device in author's laboratory. Note dial gage in place and a load of 200 kPa (4 load plates) applied.

rated or partially saturated coarse-grained soil, the coefficient of permeability k is large enough so that the pore water can also escape relatively instantaneously.

When a load is applied to a fine-grained soil which is saturated or partially saturated, the time for all the plastic deformation and void-ratio reduction to take place is much longer. The length of time for the process will depend on several factors, of which the primary ones are:

1. The degree of saturation
2. The coefficient of permeability of the soil
3. The properties of the pore fluid
4. The length of the path the expelled pore fluid must take to find equilibrium

Consolidation will now be defined as that plastic deformation void-ratio reduction (generally termed *settlement*) which is a function of time, or

$$S = f(t) \quad (13-1)$$

The state-of-art at present can formulate a mathematical model for the consolidation process (which applies with reasonable success) only in the case of a fully saturated soil.

When a load is applied to a fine-grained saturated soil confined in a metal ring with piezometers inserted in the sample (as shown in Fig. 13-2a), the water level in all the piezometers will move to a new height of $h + \Delta h$ immediately after the load Δp is applied, as shown in Fig. 13-2b.¹ The reason for this is that when a low coeffi-

¹This is fictitious since any quantity of flow sufficient for this Δh to be visualized would dehydrate the sample. The concept is essentially correct and helps the reader to comprehend the process.

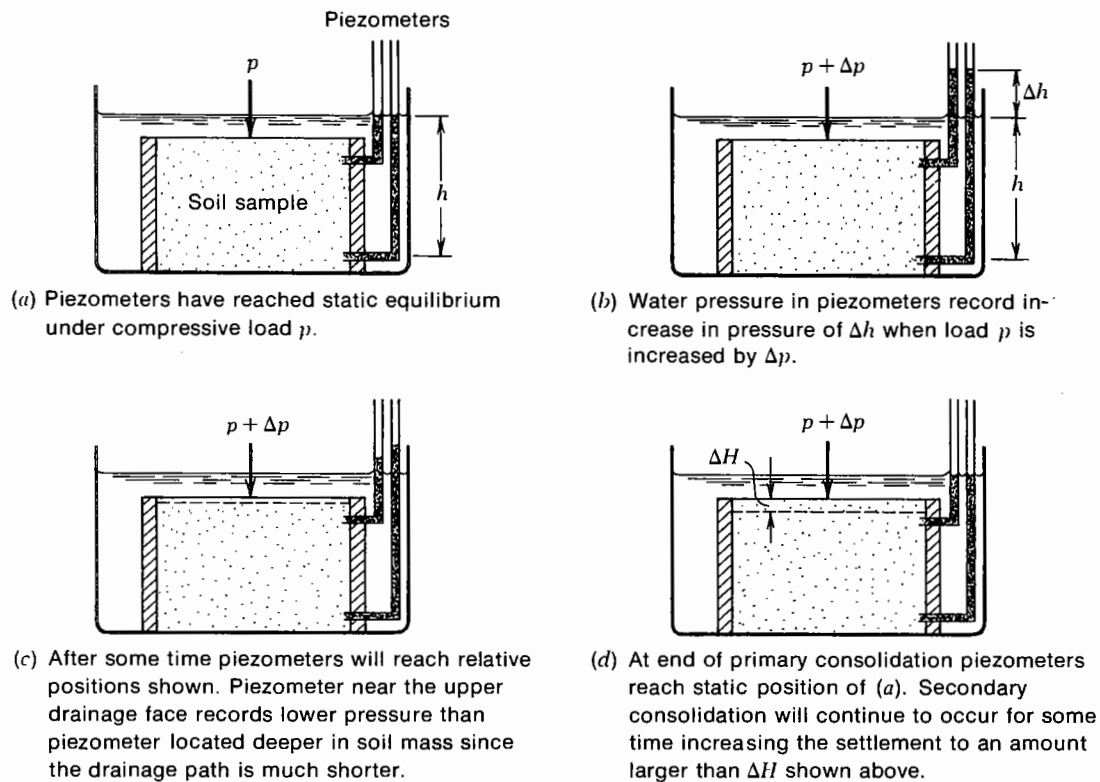


Figure 13-2
Primary consolidation

cient of permeability exists (as for the fine-grained soils such as silts and clays), the water cannot instantly flow out of the soil pores. It follows that this load induces an attempt in the soil mass to undergo a void reduction (volume change), which cannot occur unless some water is expelled from the soil pores, but time is required for the flow of water to take place since this is a rate process. Since the water does not instantly flow, the load is converted into an increase in the pore water pressure—hence the sudden jump in the piezometer height of Δh . As the water gradually flows out of the soil pores, the piezometer head drops off, as shown in Fig. 13-2c. Since water flows from high potential (high pressure) to a lower potential, and obviously the water surface is the lowest potential for this system, the water in the soil mass flows toward the surface of the water surrounding the soil mass. The length of the flow path is shorter for water in the soil pores near the surface of the soil sample (Fig. 13-2c), where the water quickly flows out, causing a more rapid head reduction in the top piezometer than in the piezometers located with longer flow paths from the free water surface. This is also evident from Darcy's equation,

$$v = ki$$

In the soil sample the water must travel some distance Δy . To travel this distance at the velocity of Darcy's equation requires some time interval, Δt , as:

$$\Delta y = \frac{k}{L} \left(\frac{dh}{dt} \right) \Delta t$$

Consolidation (also termed *primary consolidation*) is considered to be complete when the water level in all the piezometers returns to the static water level, as

shown in Fig. 13-2d. Actually, additional soil deformation continues after this time; this is termed *secondary consolidation*. No reliable mathematical model is currently available to describe the secondary consolidation of a soil, although it is known that it exists and is essentially a continuing soil-grain readjustment to the applied load.

We can now redefine consolidation as *that plastic deformation/void-ratio reduction of a soil mass which are functions of time and excess pore pressure*.

The laboratory test is *one-dimensional* in that, with a metal ring confining the sample, no lateral soil or water movement takes place—all water flow and soil movement are in the vertical direction. In the field some lateral water movement may occur, as well as some slight lateral soil movement. Neither of these effects is probably very important when considering the overall settlements due to consolidation based on extrapolating laboratory results to the field.

The laboratory consolidation test is performed on a specimen between 20 and 40 mm thick placed in a confining metal ring ranging in diameter from about 45 to 113 mm (100 cm²). Most commonly, diameters of 63 and 113 mm are used. The apparatus may be of the *fixed-ring* or *floating-ring* type. The *fixed-ring* device can be used to measure the coefficient of permeability¹ of the sample as it is being tested. The advantage of the *floating-ring* device is that it reduces the friction loss along the sides of the sample between the soil and ring (Fig. 13-3c) and the test rate is about 4 times faster. For all things equal, since the sample preparation effects are about the same for any size sample, the larger samples provide the highest percentage of reliability. In any case, the ring ID should be at least 6 mm smaller than the tube sample and the diam/height ratio > 2.5.

The Anteus consolidometer² was introduced (ca. 1964) to attempt to produce a consolidation test in which the pore pressure within the sample could be controlled. That is if a sample is from 3 m below the water table, a static pore pressure exists during consolidation of $3 \times 9.807 = 29.42$ kPa and this device is able to produce this (or some other value) pressure, termed *back pressure*.

The consolidation test proceeds by applying loads in a geometric progression with a load ratio, $\Delta p/p = 1$ with a typical load sequence as follows:

25, 50, 100, 200, 400, 800, 1600 (and sometimes 3200) kPa

An alternative loading sequence, and depending on equipment available, might be:

5, 10, 20, 40, 80, 160, ... etc., kPa

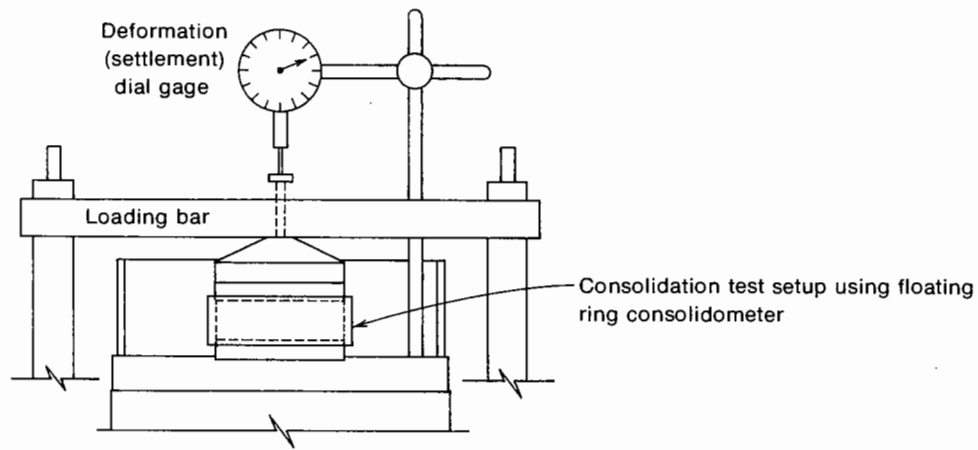
Other load increments can be used, but it appears that if the increment ratio of $\Delta p/p$ is not large enough, the soil tends to build internal resistance to the loads and the total deformation of the sample will be less than obtained if the load increment ratio of $\Delta p/p = 1$ (as illustrated above) is used.

Consolidation-test data (refer to Fig. 13-5) are obtained from a test and used as follows:

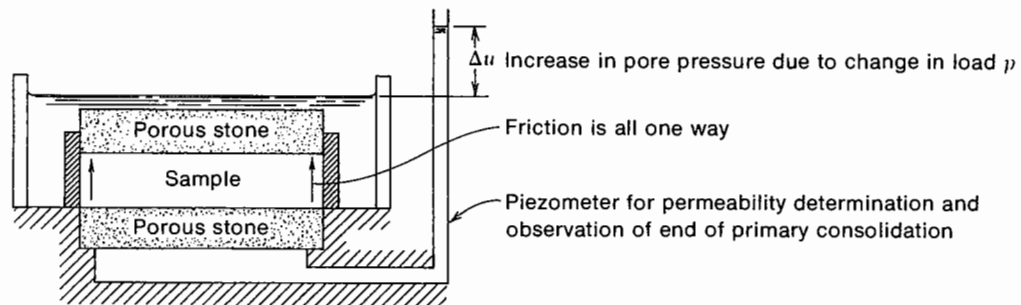
1. Weight, dimensions (height and diameter), and water content of the original

¹This is not a trivial procedure and requires setting up a falling head permeability test as in Experiment No. 12. Use a 100-ml burette with a rubber balloon to control evaporation and run a falling head test between each load increment to obtain the relationship between k vs. e .

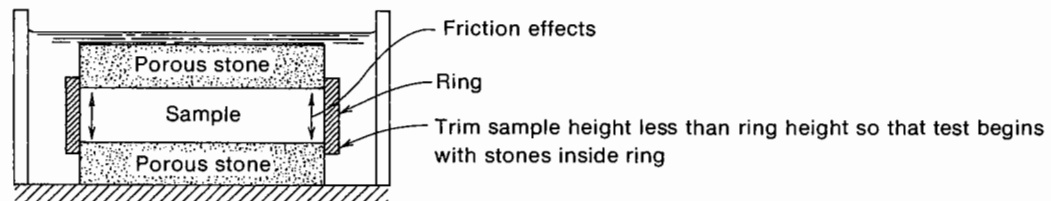
²Very few, if any, undergraduate soils laboratories use this device, therefore the reader will have to consult literature furnished with the consolidometer for operational details. The device is too complicated and difficult to use to attempt a description of its operation in this text.



(a) Consolidometer.



(b) Fixed-ring consolidometer. May be used to obtain permeability information during a consolidation test if a piezometer is installed.



(c) Floating ring consolidometer.

Figure 13-3
Line details of a typical consolidation test.

consolidation-test specimen are obtained so that the initial void ratio e_0 and cross-sectional area A can be computed and the initial height H_i of the sample can be established.

The total volume of the sample can be determined from H_i and the area A , but it can also be computed as

$$V_t = V_s + V_w \quad (13-2)$$

where V_s and V_w are the volume of soil solids and volume of soil water, respectively. The volume of water can be computed from the water content if reliably known at the beginning of the test as

$$V_w = \frac{wW_s}{G_w\gamma_w} = wW_s \quad (13-3)$$

since the specific gravity and unit weight of water can be taken as 1.00 without appreciable error.

The volume of water at the end of the test V_{wf} can be obtained directly as the loss of water due to oven-drying the sample. At this time the saturation S should be 100 percent (it may be some less at the beginning)¹. This procedure of obtaining the volume of water, and with the initial sample dimensions, the initial volume of water V_w of Eq. (13-3), is preferred by the author.

The height of the soil solids (solid mass in the block diagram) can be readily computed after oven-drying the soil cake and from the total change in height ΔH as follows:

$$\begin{aligned} H_f &= H_{\text{initial}} - \Delta H && \text{(initial sample height and use of dial readings)} \\ V_{\text{final}} &= H_f A = H_s A + V_{wf} && \text{(volume = solids + water)} \end{aligned}$$

from which, the height of solids, H_s is

$$H_s = H_f - \frac{V_{wf}}{A} \quad (13-4)$$

and A = area of consolidometer ring.

The initial height of voids can be computed using

$$H_v = H_{\text{initial}} - H_s \quad (13-5)$$

and the initial void ratio, e_i , is

$$e_i = \frac{H_v}{H_s} \quad (13-6)$$

An additional check on the computed value of H_s is obtained if the specific gravity of the soil solids is known; alternatively, we may compute G_s from H_s and the weight of soil solids W_s , and if the value is not reasonable, we may attempt some method of reconciliation between various items of information from the test to obtain a reasonable value of both H_s and G_s .

2. The amount the sample compresses under a given load as periodic time intervals elapse, is recorded as part of the data. This information is obtained by attaching a dial gage (sensitive to 0.01 mm/div—author's recommendation; ASTM suggests 0.0025 mm/div, but this is too precise as stated earlier) to take readings of sample deformation at certain time intervals. A total time interval for a sample to consolidate under a load increment may be 24 to 48 h or more and to give all the readings equal importance these data are usually presented as a semilogarithmic plot of dial reading (DR) vs. time (time on the logarithmic scale) in minutes for each load increment. The greatest amount of the total compression of the sample occurs soon after load application, and it is necessary to give this part of the curve as much importance as the end portion.

Sometimes these time-settlement data are presented as a plot of dial reading vs. $\sqrt{\text{time}}$ (again in minutes). This method of data presentation is credited to D. W. Taylor, who found that the method provided reasonably reliable results for clay soils around Boston, Massachusetts.

It should be observed that a plot of dial readings is the same as a plot of the compression of the sample vs. time since the difference between any two dial readings is the compression of the sample for the corresponding time interval.

¹Due both to sample expansion due to loss of overburden pressure and hydrostatic water pressure loss causing air to come out of solution.

From a plot of dial reading vs. log time (or $\sqrt{\text{time}}$), as shown in Fig. 13-6a and b, one may obtain D_{100} (the dial reading corresponding to 100 percent consolidation, or $U = 100$ percent), D_0 , D_{50} , and corresponding times for this to occur, t_{100} , t_{50} , etc. for each load increment.

- a. To obtain D_{100} (which is arbitrarily defined) from the DR vs. log time curve, draw tangents to the middle and end parts of the curve, as shown in Fig. 13-6a. At the intersection of the tangents, project horizontally to the curve ordinate to read D_{100} . To obtain t_{100} (the time at which this amount of compression occurs), project horizontally from the tangent intersection to the curve, then vertically down to the abscissa for the time value. This method is quite arbitrary but is widely, if not universally, used. It is based on the assumption that the end part of the curve defining secondary compression is linear as is the interior part of the primary consolidation curve, thus, the curve discontinuity is at D_{100} .
- b. To obtain D_0 (the theoretical dial reading at $t = 0$ since one cannot plot the logarithm of time for $t = 0$) on the semilogarithmic plot, *if the early part of the curve is parabolic*, select a time t_1 and a time $t_2 = 4t_1$. Measure the ordinate y from t_1 to t_2 on the curve and lay this same value of y off vertically above t_1 . Draw a horizontal line through this point and call the intercept of this line on the DR ordinate D_0 (Fig. 13-6a). Slightly refined results may be obtained by using other points along the curve for t_1 and t_2 and taking the horizontal line through the average of the y values thus obtained. If this value of D_0 is considerably different from the actual dial reading at t_0 (which is what D_0 is supposed to represent) or if the early part of the curve is not parabolic, use the actual dial reading at $t = 0$ for D_0 . With D_0 and D_{100} established, one may obtain the dial reading corresponding to 50 percent consolidation D_{50} as

$$D_{50} = \frac{D_0 + D_{100}}{2} \quad (13-7)$$

- c. To make a plot of dial reading vs $\sqrt{\text{time}}$, obtain the time corresponding to a given dial reading; i.e., if $t = 25$ min, $\sqrt{\text{time}} = \sqrt{25} = 5$, which is then plotted against the dial reading for 25 min. This is done for the remainder of the time and dial-reading data. To find D_0 , Taylor recommended drawing a straight line through the first several (say, 6 to 8) plotted points and extending this line until it intersects the ordinate, as shown in Fig. 13-6b (line A). This intersection of the ordinate establishes D_0 .

Next, take an abscissa value 15 percent greater than the value obtained by the intersection of the straight-line plot of DR versus $\sqrt{\text{time}}$ and the time axis, and from D_0 draw a straight line through this point (as line B of Fig. 13-6b). Continue plotting values of DR vs. $\sqrt{\text{time}}$ and draw a smooth curve using a French curve for the curved part of the plot. Where the plotted curve crosses the line B (1.15 offset line), the ordinate value is arbitrarily taken as D_{90} . We can now compute D_{100} as

$$D_{100} = D_0 - \frac{10}{9}(D_0 - D_{90}) \quad (13-8)$$

With D_{100} now known, D_{50} can be obtained using Eq. (13-7).

- d. The value of t_{50} or $\sqrt{\text{time}}_{50}$ can be found from the DR vs. time curve by entering with the value of D_{50} from Eq. (13-7). Other D values and the corresponding time values can be found in a similar manner for each curve (for each load increment). The sole reason for finding D_0 is to determine the time value for various percentages of consolidation (often the symbol U is used).

The time at 50 percent consolidation t_{50} (usually) or some other time will be used to find the *coefficient of consolidation* c_v :

$$c_v = \frac{TH^2}{t} \quad (13-9)$$

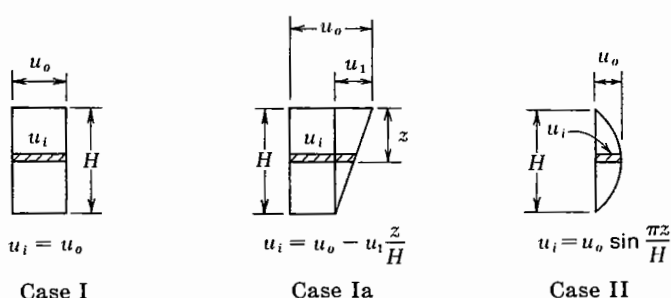
where T = time factor (0.197 for $U = 50$ percent), given in Table 13-1 for two pore-pressure cases

t = time for the corresponding time factor T (if $T = 0.197$, use the elapsed time corresponding to $t = t_{50}$)

H = average length of the longest drainage path during the given load increment (with a porous stone on both faces of the sample, $H = \frac{1}{2}$, the average sample height during the application of the given load increment)

Table 13-1 Time Factors for Indicated Pressure Distribution

$U(\%)$	Case I	Case II
0	0.000	0.000
10	0.008	0.048
20	0.031	0.090
30	0.071	0.115
40	0.126	0.207
50	0.197	0.281
60	0.287	0.371
70	0.403	0.488
80	0.567	0.652
90	0.848	0.933
100		



Pore-pressure distribution for case I usually assumed for case Ia.

The coefficient of consolidation is plotted as c_v versus log pressure p , generally on the same graphic plot as the void ratio vs. log pressure at a different ordinate (natural) scale. Some persons plot this as an average of load increments; however, the curve is usually so erratic because of limitations in the theory, and because of the method of obtaining H in Eq. (13-9), it seems reasonable to plot c_v against the corresponding load increment as shown in Fig. 13-9.

- e. The slope of the secondary branch of the settlement vs. log time curve is related to creep or *secondary consolidation*. This slope is approximately constant for a given soil and this fact can be used to obtain D_{100} for load increments which have been changed while the sample was just at the end of primary consolidation. The slope of the secondary branch of the curve can

be used to compute a secondary compression coefficient, C_α , for secondary compression estimates as (see Fig. 13-6a):

$$C_\alpha = \frac{\Delta H}{\log t_2/t_1} \quad (13-10)$$

3. Plotted relationships may be made between:

Void ratio vs. pressure

Strain vs. pressure

a. *Plot of void ratio e vs. pressure.* This plot may be made using either log pressure (Fig. 13-9) or as an arithmetic plot (Fig. 13-8).

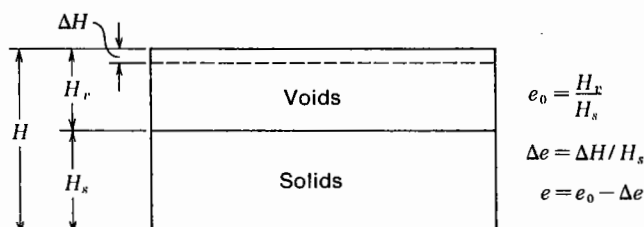


Figure 13-4
Void-ratio change in height relationship.

From Fig. 13-4 it is evident that while the soil skeleton deforms, this results in a loss of voids, the total solids remaining constant. Thus, the initial void ratio, e_i , is

$$e_i = \frac{H_v A}{H_s A} = \frac{H_v}{H_s}$$

At any other load increment, the instant void ratio e is

$$e = \frac{(H_v - \Delta H)A}{H_s A} = e_i - \frac{\Delta H}{H_s}$$

or $e = e_i - \Delta e$

The change in height ΔH is obtained from the changes in dial readings for the load increments, thus, the change in void ratio is readily computed.

From an arithmetic plot of e versus pressure one can obtain the *coefficient of compressibility* a_v (Fig. 13-8) as:

$$a_v = \frac{\Delta e}{\Delta p} \quad (\text{usually disregard negative sign}) \quad (13-11)$$

and the *coefficient of volume compressibility* m_v as:

$$m_v = \frac{a_v}{1 + e_0} \quad (13-12)$$

Note that the unit of m_v is the reciprocal of the modulus of elasticity and inspection of a_v and m_v indicates that if we interpret

$$\frac{\Delta e}{1 + e_0} = \frac{\Delta L}{L} = \epsilon \text{ (strain)}$$

and p = pressure

$$E = \frac{\Delta p}{\epsilon} \text{ or } \frac{1}{E} = \frac{\epsilon}{\Delta p} = m_v$$

From the semilog plot (most common method of presenting data) of void ratio vs. log pressure (as Fig 13-9), we obtain, from the straight-line part, the *compression index* C_c as

$$C_c = \frac{\Delta e}{\log p_2/p_1} \quad (\text{neglect negative sign}) \quad (13-13)$$

and from the unload branch of the curve obtain the *swell index* C_s as

$$C_s = \frac{\Delta e_s}{\log p_2/p_1} \quad (13-14)$$

The *recompression index* C_r is obtained from the recompression branch as:

$$C_r = \frac{\Delta e_s}{\log p_2/p_1} \quad (13-15)$$

Note that in all three of these equations if the slope is extended across 1 log cycle, the $\log p_2/p_1$ ratio = $\log 10 = 1$ and it is only necessary to obtain the value of Δe for 1 cycle of the plot for the numerical value of the coefficient.

B. *Plot of strain ϵ vs. log pressure.* This method of plotting data is relatively recent and has the particular advantage of plotting the data as the test progresses. The typical plot is in Fig. 13-10 and is identical in shape to the e vs. $\log P$ plot of Fig. 13-9. The slope of the straight line part of the curve is called the *compression ratio* C'_c defined as:

$$C'_c = \frac{\Delta \epsilon}{\log p_2/p_1} \quad (13-16)$$

The slope of the recompression branch may be called the *recompression ratio* C'_r and the slope of the swell branch may be called the *swell ratio* C'_s with values obtained analogous to C_r and C_s .

4. In general, use the initial and final dial readings for each load increment to compute either strain or change in void ratio. Use D_0 and D_{100} to obtain t_{50} for computation of the coefficient of consolidation.
5. When the void ratio vs. log pressure or strain vs. log pressure curve has been drawn, it will be found that if the test has been performed on an undisturbed sample from the field, it will have a characteristic curve such as point 0 of Fig. 13-10. This characteristic shape is attributed to unloading the sample of its overburden pressure when it is collected from the field. This conclusion has been made from observing a similar curve shape being obtained by unloading a consolidation-test sample in the laboratory and then reloading it as illustrated in Fig. 13-9.

From this observation, Casagrande (1936) proposed that the preconsolidation pressure could be estimated as follows (see Fig. 13-9):

- a. At the sharpest part of the curve, estimated by eye, draw a tangent.
- b. Through this point of tangency, draw a horizontal line such as line $O-C$ of Fig. 13-10.
- c. Bisect the angle formed in steps a and b above.
- d. Extend the straight line of the e - $\log p$ (or ϵ - $\log p$) curve until it intersects the angle-bisector line.

- e. Drop a vertical line to the abscissa and read the corresponding value of pressure p . If $p = p_0$, the computed overburden pressure, the soil is *normally consolidated*. If $p > p_0$, the soil is *preconsolidated* and this value of p should be identified as p_c , the preconsolidation pressure.
- f. On the e -log p curve the in situ void ratio e_0 can be estimated as:
 - (1) Normally consolidated soil: On the e -log p curve at p_0 .
 - (2) Preconsolidated soil: At the intersection of lines defining p_c (as point B of Fig. 13-10 used for illustration of location) as shown in Fig. 13-9. Carefully note that the initial void ratio of the test specimen e_i is generally larger than the in situ void ratio e_0 although in much of the literature a clear cut distinction is not made between the two values.

Procedure *This Will Be a Class Project*

1. Carefully trim a sample to fit the consolidation ring (or use the sample provided by the instructor).¹

Optional: Collect a representative sample of the trimmings and use for a water-content sample. Determine the specific gravity G_s .

Weigh the sample and determine the sample height H_i and the diameter of sample. *Note:* After trimming the sample to the nominal dimensions (and/or using the spacer disk), record the thickness to the nearest 0.01 mm (0.0001 in) i.e., 20.00 mm (0.8000 in) or other dimension.

2. Carefully place the soil sample in the consolidometer with a *saturated* porous stone on each face. Be sure the porous stones will enter the ring so that the test can proceed satisfactorily.

Place consolidometer into the loading device and attach the dial gage; remember to allow for possible sample compression of 4 to 12 mm.

Apply a seating load of say 5 (for soft) to 10 (for stiff soil) kPa. Again check that the porous stones do not hang on the ring. Now zero the deformation dial gage (leave the seating load on the soil).

3. At a convenient starting time, apply the first load increment (sufficient additional load to develop first load increment) and simultaneously take deformation readings at elapsed times of

0.25, 0.50, 1, 2, 4, 8, 15, 30, 60, 120 min, then, say, 4, 8, 16 h, etc.

of elapsed time and until there is little change (plot DR vs. $\log t$) in the curve.

For student laboratories and if a quick consolidating sample is used, modify the reading sequence to change loads at 30, 60, or 90 min or as soon as a plot of the data indicates sample is well within the secondary branch of the consolidation curve.

4. After 24 h or as directed, or when the ΔH between two readings is sufficiently small, change the load to the next value and again take elapsed-time interval readings as in step 3, above.

If using a "quick" laboratory test, on the third or fourth load increment, take the test sufficiently long to establish a valid secondary consolidation slope for use on all the other load increments.

5. Continue changing loads and taking elapsed time vs. deformation dial readings through the load range of the consolidometer (or until arbitrarily terminated).

¹The author recommends preparing a sample of fine, silty sand which will consolidate in 30 min to 1 h so that the test can be performed in 1 day. The sample will not have a preconsolidation pressure, but all other aspects of the consolidation test can be observed. Advanced classes should perform the test in a realistic manner.

6. Place the sample (including all the squeezed-out particles) in the oven at the end of the test to find the weight of soil solids W_s and to enable a computation of the final volume of water V_{wf} . Compare W_s with the computed value from step 1 (if a water-content determination was made).
7. Plot curves of dial reading vs. log time. Find D_0 , D_{100} , D_{50} and the corresponding t_{50} for each load increment and show the values on all curves. Obtain 5-cycle semilog paper from your data sheet section—plot more than one load increment on a sheet but do not excessively bunch the plots. Refer to Figs. 13-5 to 13-10 for computations and method of displaying data.
8. *Optional*: Plot dial readings vs. $\sqrt{\text{time}}$ for any two load increments and find D_0 , D_{100} , D_{50} and the corresponding t_{50} . Compare t_{50} to that obtained in step 7 above and make appropriate comments in your report if there is a large discrepancy.
9. Using your time vs. dial reading sheets, your computations for e_0 , H_s , complete the "Computation sheet for e and c_v " data sheet in your data sheet set (refer to Fig. 13-7).
10. Plot either a or b or both as directed by instructor.
 - a. Compute the strain for each load increment as

$$\epsilon = \frac{\Sigma \Delta H}{H_i}$$

where $\Sigma \Delta H$ is the accumulated deformation through any load increment. Plot ϵ vs. $\log P$ and obtain the compression ratio C'_c , and compute the compression index as

$$C_c = C'_c(1 + e_0)$$

and show both values on your ϵ vs. $\log P$ plot.

- b. Compute e_0 , e_i and make a plot of e vs. $\log P$ and compute the compression index, C_c . Show this computation on the curve sheet together with how you obtained the values for the computation (curve coordinates).
11. On the curve from step 10, select a good ordinate scale and plot c_v vs. $\log P$ using the right side of the graph sheet to set the c_v scale. Do not use such a large scale that the e (or ϵ) vs. $\log P$ curve is overwhelmed. Be sure to use a scale which reasonably indicates $c_v = \text{constant}$ (which is an essential assumption in consolidation theory).
12. From the e (or ϵ) vs. $\log P$ curve find the preconsolidation pressure (if applicable).
13. Prove that $C'_c = \frac{C_c}{1 + e_0}$.

CONSOLIDATION TEST

Data Sheet 14

Project Lab Manual Data Job No. ~
 Location Bradley University Boring No. ~ Sample No. ~
 Description of Soil Blue Clay w/very fine sand Depth of Sample ~
 $w_L = 35.7\%$ $w_P = 19.6\%$
 Tested By K.M. & M.C. Date of Testing 11/23-29/76
 Consolidometer type Floating Ring Ring no. SI-1
 Multiplication ratio of load device 27:1 (Hydraulic type)
 Ring dimensions: Diam. 62.14 mm Area, A 30.33 cm² Ht. 24 mm

Initial ht. of soil, H_i 20.00 mm

Specific gravity of soil, G_s = ~

Wt. of ring + specimen at beginning of test = 292.02 g

Wt. of ring = 164.90

Wt. of wet soil, W_t = 127.12

Computed dry weight of soil, W_s = ~

Oven dry wt. of soil, W_s = 99.70 g

Water content determination

Wt. of can + wet soil = ~

Wt. of can + dry soil = ~

Wt. of can = ~

Wt. of water = ~

Wt. of dry soil = ~

Initial water content w_i = ~

Computed Ht. of solids^b H_s = $[2(30.33) - (0.364 \times 30.33 + 16.75)] / 30.33 = 1.084 \text{ cm}$

Initial Ht. of voids, $H_v = H_i - H_s$ = $2.000 - 1.084 = 0.916 \text{ cm}$

Initial degree of saturation, $S_i = (W_t - W_s) / (H_v A)$ = $(127.12 - 99.70) / (0.916 \times 30.33) = 0.987$

Initial void ratio $e_i = H_v / H_s$ = $0.916 / 1.084 = 0.845$

Final Test Data (obtained at end of load testing)

Initial dial reading 0.00

Final dial reading $364 \times 0.01 = 3.64 \text{ mm}$

Change in sample ht. 0.364 cm

Final ht. of voids, $H_{vf} = \frac{16.75}{30.33} = 0.552 \text{ cm}$

Final void ratio, $e_f = H_{vf} / H_s$ 0.509

Final water content determination

Final wet wt. + ring^c 281.35 g

Final dry wt. + ring 264.60
ring 164.90

Oven dry wt. of soil, W_s 99.70 g

Final water content, w_f $\frac{16.75}{99.70} \times 100 = 16.8\%$

Final degree of sat. S 100 (assumed) %

^aObtained from Final Water-Content data.

^bUse either G_s or final water-content data for $S = 100\%$.

^cBe sure to include any soil extruded from ring which is in consolidometer.

Figure 13-5

Consolidation test data.

(a) Typical preliminary data from consolidation test.

CONSOLIDATION TEST (Time-compression data)

Data Sheet 15

Project Lab manual Data Job No. ~
 Location of Project Bradley University Boring No. ~ Sample No. ~
 Description of Soil Blue Clay w/very fine sand Depth of Sample ~
 Tested By K.M. & M.C. Date of Testing Nov 23-29, 1976

Loading Test Data

Load 100 kPa

Load 200 kPa

Date applied 11/25/76

Date applied 11/26/76

Applied by K.M.

Applied by M.C.

Clock time and date	Elapsed time, min	*Dial readings $\times 10^{-2}$ mm/div	
		Original	Adjusted
<u>11-25</u>			
<u>8:27</u>	<u>0</u>	<u>59</u>	<u>0.59 mm</u>
	<u>0.1</u>	<u>66</u>	
	<u>0.25</u>	<u>67.5</u>	
	<u>0.5</u>	<u>69.5</u>	
	<u>1</u>	<u>70</u>	
	<u>2</u>	<u>72</u>	
	<u>4</u>	<u>73</u>	
	<u>8</u>	<u>75</u>	
	<u>15</u>	<u>78</u>	
	<u>30</u>	<u>80</u>	
	<u>60</u>	<u>83.5</u>	
<u>10:27</u>	<u>120</u>	<u>88.5</u>	
<u>12:43</u>	<u>256</u>	<u>94.0</u>	
<u>3:27</u>	<u>420</u>	<u>97</u>	
<u>11-26</u>			
<u>8:04</u>	<u>1417</u>	<u>98</u>	<u>0.98 mm</u>

Clock time and date	Elapsed time, min	Dial readings $\times 10^{-2}$	
		Original	Adjusted
<u>11-26</u>			
<u>8:04</u>	<u>0</u>	<u>98</u>	
	<u>0.1</u>	<u>109.5</u>	<u>0.11 mm</u>
	<u>0.25</u>	<u>112</u>	
	<u>0.5</u>	<u>113.5</u>	
	<u>1</u>	<u>116</u>	
	<u>2</u>	<u>117.5</u>	
	<u>4</u>	<u>120</u>	
	<u>8</u>	<u>123.5</u>	
	<u>15</u>	<u>127</u>	
	<u>30</u>	<u>132.5</u>	
	<u>60</u>	<u>138</u>	
<u>10:04</u>	<u>120</u>	<u>143.5</u>	
<u>12:00</u>	<u>256</u>	<u>148</u>	
<u>5:43</u>	<u>579</u>	<u>151</u>	
<u>11-27</u>			
<u>7:34</u>	<u>1410</u>	<u>152</u>	<u>1.52 mm</u>

*Insert gage subdivisions 0.01 mm/div, etc.

(b) Data to plot dial reading vs. time curves shown in Fig. 13-6.

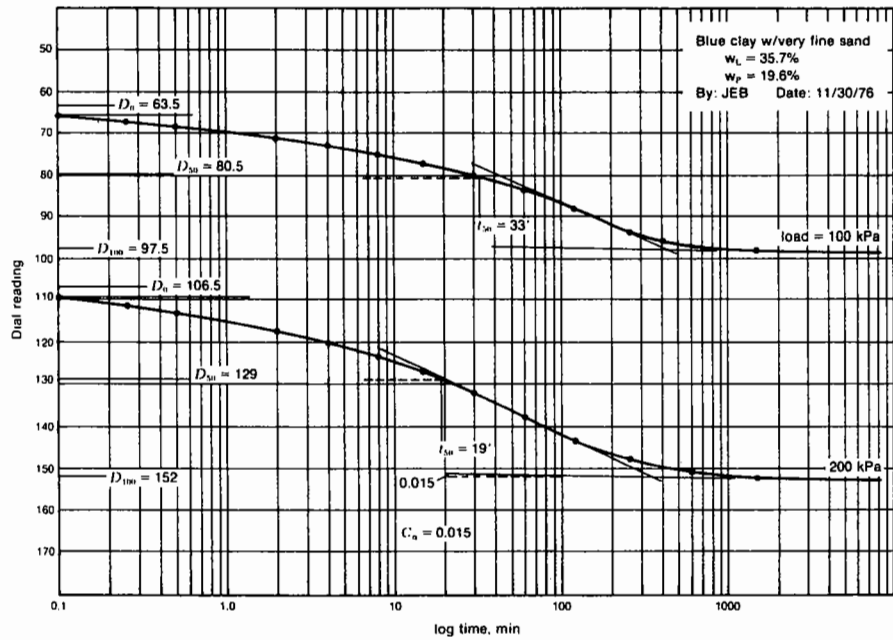
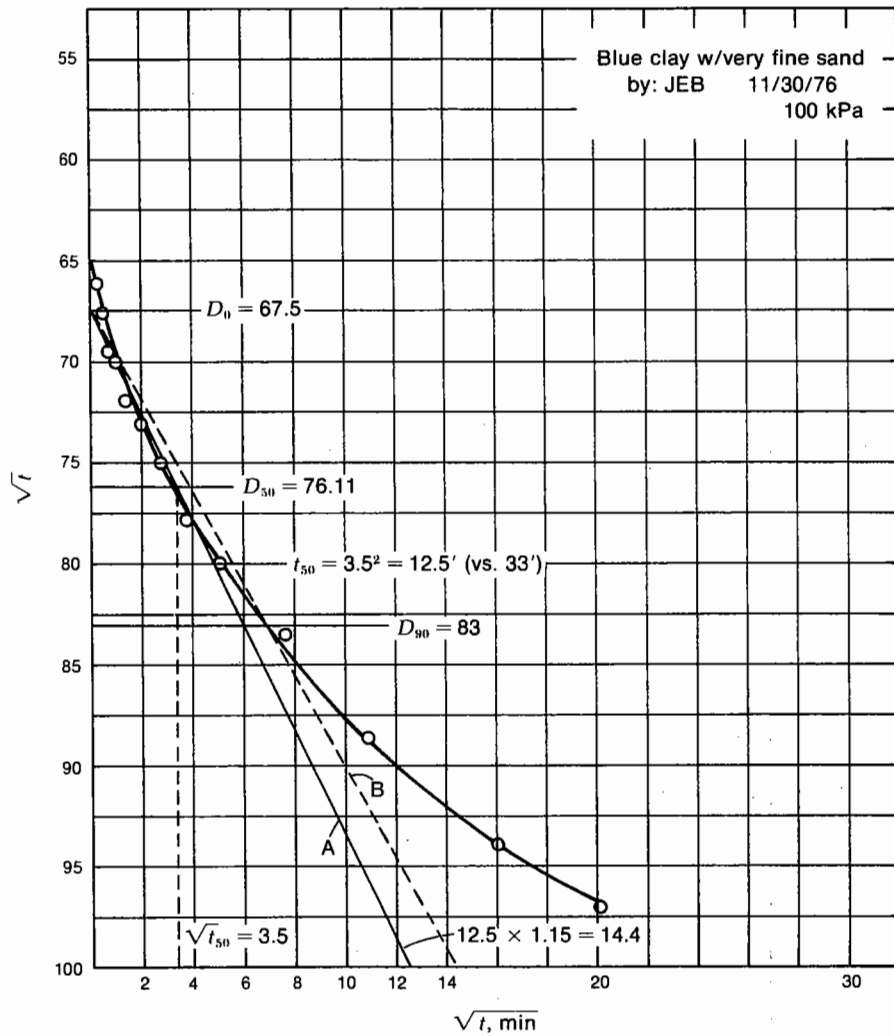


Figure 13-6
 Dial reading vs. time
 curves.
 (a) Dial reading vs. log
 time (min).



(b) Dial reading vs.
 $\sqrt{\text{time}}$ (min).

Sample Computations

From line 2 of Fig. 13-9 (load = 25 kPa)

Initial dial reading = 0.00 cm

Final dial reading = 0.032 cm ($32 \times 0.01 \times 0.1$)

$$\Delta H = 0.032 \text{ cm}$$

$$\Delta e = \frac{H}{H_s} = \frac{0.032}{1.084} = 0.0296 \rightarrow 0.030 \text{ (plotting accuracy)}$$

$$e = e_i - \Delta e = 0.845 - 0.030 = 0.815$$

$$\epsilon = \frac{\Delta H}{H_i} = \frac{0.032}{2.00} = 1.6 \times 10^{-2}$$

$$\text{Av. height} = H_i - \frac{(\Delta H_{i-1} + \Delta H_i)}{2}$$

$$= 2.00 - \frac{0.00 + 0.032}{2} = 1.984 \text{ cm}$$

$$c_v = \frac{0.197H^2}{t_{50}} = \frac{0.197(1.984/2)^2}{10.0} = 0.0194 \text{ cm}^2/\text{min (2-way drainage)}$$

From line 4 (load = 100 kPa)

Initial dial reading = 59 = 0.059 cm

Final dial reading = 98 = 0.098 cm

$$\Delta H = 0.098 \text{ cm}$$

$$\Delta e = \frac{0.098}{1.084} = 0.090$$

$$e = 0.845 - 0.090 = 0.755$$

$$\epsilon = \frac{0.098}{2.00} = 4.9 \times 10^{-2}$$

$$\text{Av. height} = 2.00 - \frac{0.059 + 0.098}{2} = 1.922 \text{ cm}$$

$$c_v = \frac{0.197(1.922/2)^2}{33.0} = 0.0055 \text{ cm}^2/\text{min}$$

CONSOLIDATION TEST (Computation sheet for e and c_v)

Data Sheet 16

 Project Lab Manual Data Job No. ~

 Location of Project Bradley University Boring No. ~ Sample No. ~

 Description of Soil Blue Clay w/very fine sand Depth of Sample ~

 Tested By _____ Date of Testing 11/30/76
Sample Data

$$H_c = 20.00 \text{ mm}$$

 Initial sample vol. $V_i = 60.66 \text{ cm}^3$ Dry wt. of soil solids, $W_s = 99.70 \text{ g}$

 Specific gravity of soil, $G_s = 3.03 \text{ (Comp.)}$ Ht. of solids, $H_s = 1.084 \text{ cm}$

 Initial ht. of voids, $H_v = 0.916$ Initial void ratio, $e_i = 0.845$

Load Incr.	Def. dial reading at end of load, ^a (—)	Change in sample ht., ΔH , (cm)	$\epsilon = \Delta H/H_i$ or $e = \Delta H/H_s$	Inst. void ratio e	Average ^c ht. for load, (cm)	H , ^d (cm)	Time ^e for 50% consol. t_{50} , min	Coeff. of consol. c_v , (cm ²)/min
0	0	0		0.845	—	—	—	—
25 kPa	32	0.032	1.6% 0.030	0.815	1.984	0.992	10'	0.0194
50	59	0.059	3.0 0.054	0.791	1.954	0.977	22	0.0089
100	98	0.098	4.9 0.090	0.755	1.922	0.961	33	0.0055
200	152	0.152	7.6 0.140	0.705	1.875	0.938	19	0.0091
400	223	0.223	11.2 0.206	0.639	1.812	0.906	15	0.0108
800	296	0.296	14.8 0.273	0.572	1.741	0.871	14	0.0107
1600	364	0.364	18.2 0.335	0.510 ^v	1.670	0.835	7.4'	0.0186
^a $\epsilon \times 10^{-2}$								
Rebound & recompression data (plotted on Fig. 13-9) not shown here.								
ϵ vs. $\log p$ plotted on Fig. 13-10.								

Note: Insert units in column headings as necessary.

^aFinal dial reading of preceding load = Initial dial reading of following load.

^bUse either strain or void ratio computation (or both).

^cAverage ht. for load increment = Ht. at beginning of load $-\frac{1}{2} \Delta H$.

^d H = length of longest drainage path; for floating ring consolidation = $\frac{1}{2}$ average ht. for the given load increment.

^eFrom the dial reading vs log t curves.

Figure 13-7

 Typical computations using a computation summary sheet. These computations are used to plot e vs. p , e vs. $\log p$, ϵ vs. $\log p$, and c_v vs. $\log p$ curves. The original data included an unload-reload test to compute the final void ratio of 0.497 shown on Fig. 13-9. Data has been edited for illustrative purposes to show the initial loading up to 1600 kPa.

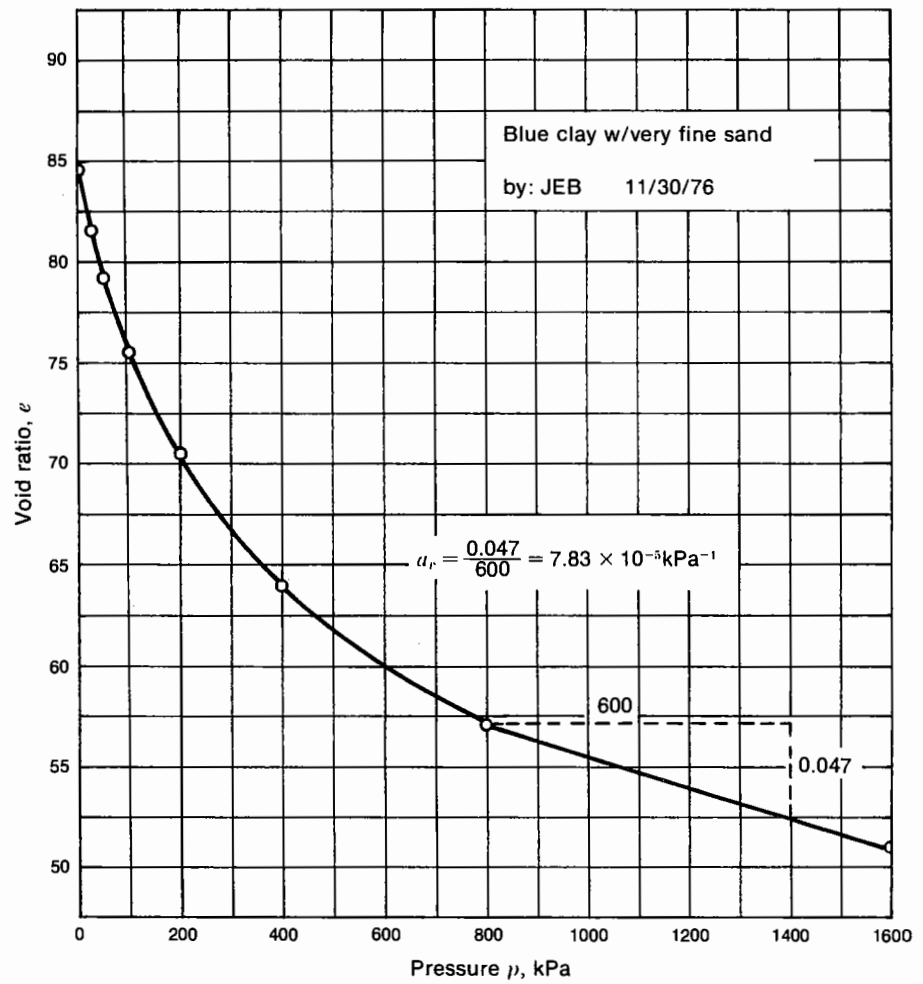


Figure 13-8
Plot of e vs. p (kPa) from
data of Fig. 13-7.

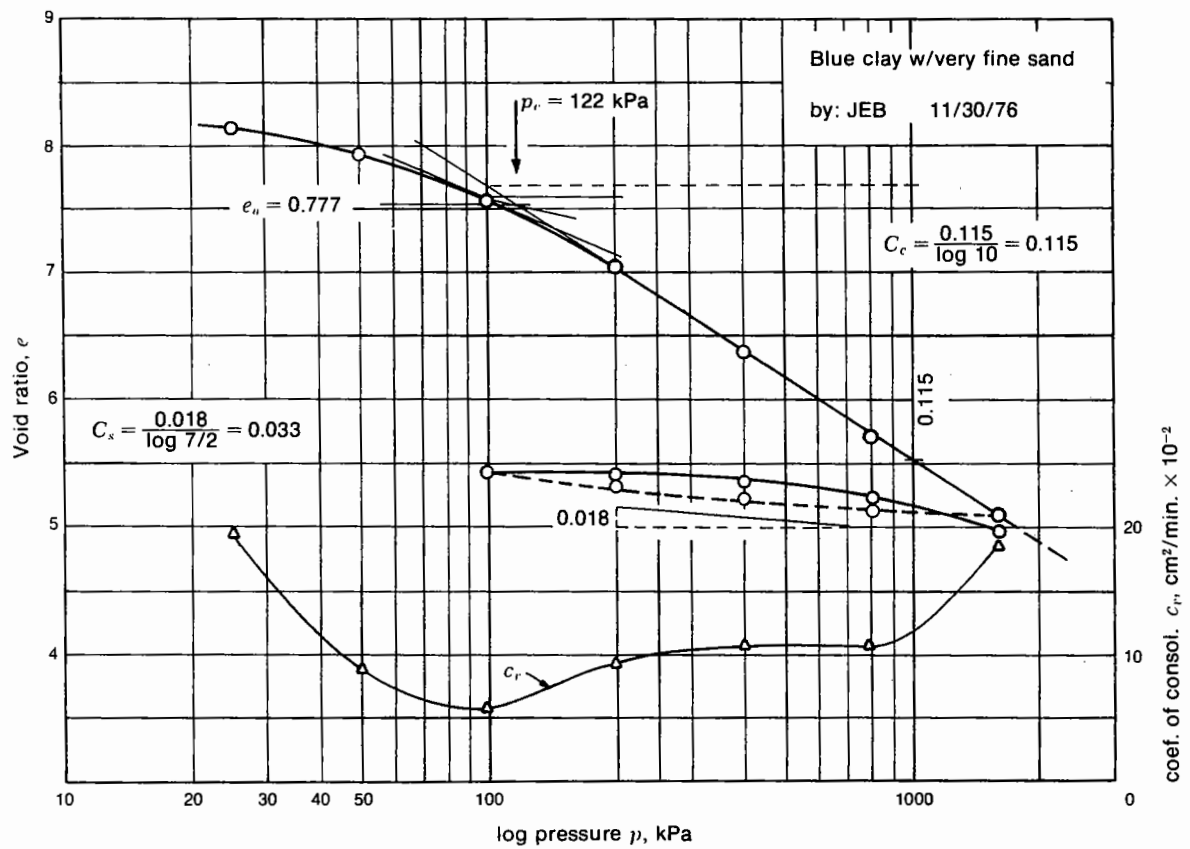


Figure 13-9
Plot of e vs. $\log p$ and c_r vs. $\log p$ using data of Fig. 13-7.

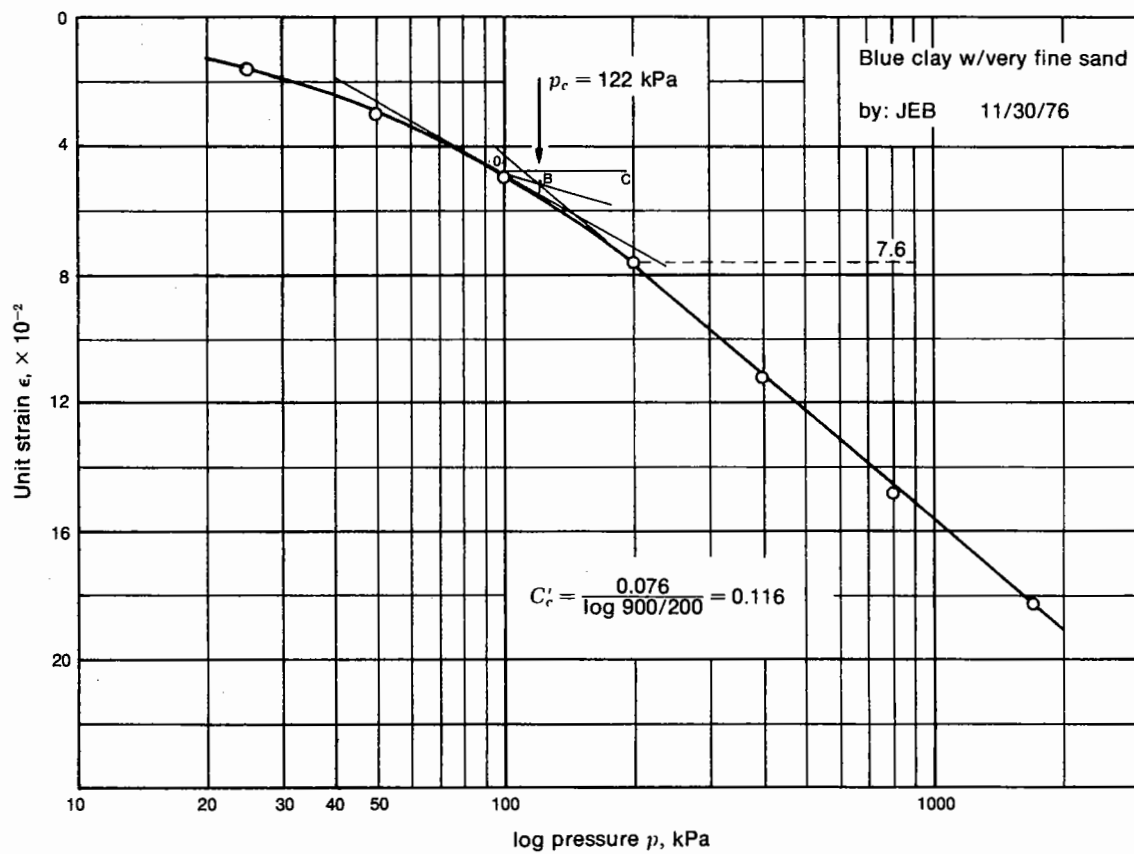


Figure 13-10
Plot of strain ϵ vs. $\log p$ using data of Fig. 13-7.

Experiment No. 14

UNCONFINED COMPRESSION TESTING

References

AASHTO T208-70
ASTM 2166-66

Objective

To introduce the student to an approximate procedure for evaluating the shear strength of a cohesive soil.

Equipment

Unconfined compression testing machine (any type of compression machine which has a load-reading device of a low enough range to obtain accurate load readings)
Dial gage (reading to 0.01 mm/div)



Figure 14-1

Unconfined compression machine. Machine may be hand-operated or motorized as here. The CBR system of Fig. 19-3 or similar can be used also to perform an unconfined compression test.

General Discussion

When the method of testing tube-recovered cohesive soil samples in simple compression was first introduced, it was widely accepted as a means of rapidly evaluating the shear strength of a soil. From a Mohr's circle construction, it is evident that the shear strength or cohesion (symbol c) of a soil sample can be approximately computed as

$$c = \frac{q_u}{2}$$

(14-1)

where q_u is always used as the symbol for the unconfined compressive strength of the soil. This computation is based on the fact that the minor principal stress σ_3 is zero (atmospheric), and the angle of internal friction ϕ of the soil is assumed zero. This $\phi = 0$ condition is the same as obtained in the unconsolidated-undrained compression test of Experiment No. 15 on a saturated soil; thus, to give the unconfined compression test more dignity, it is often called an "undrained" or U test. With more knowledge concerning soil behavior available, it became evident that the unconfined compression test does not generally provide a very reliable value of soil shear strength for at least three reasons:

1. The effect of lateral restraint provided by the surrounding soil mass on the sample is lost when the sample is removed from the ground. There is, however, some opinion that the soil moisture provides a surface-tension (or confining) effect so that the sample is somewhat "confined." This effect should be more pronounced if the sample is saturated or nearly so. This effect will depend also on the relative humidity of the testing area making a quantitative evaluation of it rather difficult.
2. The internal soil conditions (the degree of saturation, the pore water pressure under stress deformation, and the effects of altering the degree of saturation) cannot be controlled.
3. The friction on the ends of the sample from the loading platens provides a lateral restraint on the ends which alters the internal stresses an unknown amount.

Errors from the first two factors cited above can be eliminated or at least reduced by using the confined (or *triaxial*) compression tests of the next experiment. The third item has undergone considerable research, and the indication is that this factor is not as important as one might at first suppose. Special end plates, or platens, can be fabricated to reduce the friction effects if more refined test results are desired.

The unconfined compression test is widely used for a quick, economical means of obtaining the approximate shear strength of a cohesive soil. In passing, it should be noted that while the results of the unconfined compression test may not be highly reliable, there are few test methods which provide much better results unless the testing effort and procedures (of the laboratory technician) are considerably refined. The shear-strength results from the unconfined compression test are reasonably reliable if the results are properly interpreted and it is realized that the test has certain deficiencies. For example, the use of a stress-strain curve based on the unconfined compression test to obtain a modulus of elasticity (more correctly, a strain modulus since soil is not an elastic material for the usual strains associated with this type of testing) will give, in general, a very unreliable value.

The unconfined compression test may be either *strain-controlled* or *stress-controlled*. The strain-controlled test is almost universally used as it is simply a matter of attaching the proper gear ratio to a motor to control the rate of advance of a loading head. The test has been found to be somewhat sensitive to the rate of strain, but a strain rate between $\frac{1}{2}$ and 2 percent/min (i.e., a 50-mm specimen at a 1 percent strain rate would be compressed at the rate of 0.50 mm/min) appears to yield satisfactory results. Since the unconfined-compression-test specimens are exposed to the usually dry laboratory air (low humidity), they should reach failure within about 10 min; otherwise, the change in water content may effect the unconfined compressive strength (increase it, usually).

A stress-controlled test requires incremental changing of loads and may

result in erratic strain response and/or the ultimate strength falling between two stress increments. The loads are applied through a dead-load apparatus/yoke and the actual load may be the adding of water to a container or stacking of weights on a hanger. Both methods produce "shock" loading to the sample, are difficult to apply, and for these several reasons, stress-controlled tests are seldom used in any type of soil testing.

The soil specimens (and obviously only ϕ - c , or cohesive soils of apparent $\phi = 0$, can be tested in this manner) are tested until the load on the specimen begins to decrease or until at least 20 percent strain has been reached [for a specimen 76 mm long, a 20 percent strain amounts to a total axial shortening of $0.20(76) = 15.2$ mm.].

Calculations are made of axial strain and stress so that a stress-strain curve can be drawn to obtain the maximum stress (unless the stress at 20 percent strain occurs first), which is taken as the unconfined compressive strength q_u of the soil. The stress-strain curve is drawn to obtain an "averaged" value of q_u rather than merely taking the largest value of stress from the computation sheet. The unit strain ϵ is computed from mechanics of materials as

$$\epsilon = \frac{\Delta L}{L_0} \quad \text{mm/mm} \quad (14-2)$$

where ΔL = total sample deformation (axial), mm
 L_0 = original sample length, mm

The instantaneous test stress σ on the sample is computed as

$$\sigma = \frac{P}{A'} \quad (\text{kPa}) \quad (14-3)$$

where P = load on the sample at any instant for a corresponding value of ΔL , kN
 A' = cross-sectional area of specimen for the corresponding load P , m^2

It is conventional practice in soil mechanics to correct the area on which the load P is acting. This is not done when testing metal specimens in tension. One of the reasons for this correction of area is to make some allowance for the way the soil is actually being loaded in the field. Applying a correction to the original sample area is also somewhat conservative since the ultimate strength computed in this way will be less than that computed using the original area. The original area A_0 is corrected by considering that the total volume of the soil is unchanged. The initial total soil sample volume is

$$V_T = A_0 L_0 \quad (14-4)$$

but after some change in specimen length of ΔL

$$V_T = A' (L_0 - \Delta L) \quad (14-5)$$

Equating Eqs. (14-4) and (14-5), canceling terms and solving for the corrected area A' , we obtain

$$A' = \frac{A_0}{1 - \epsilon} \quad (14-6)$$

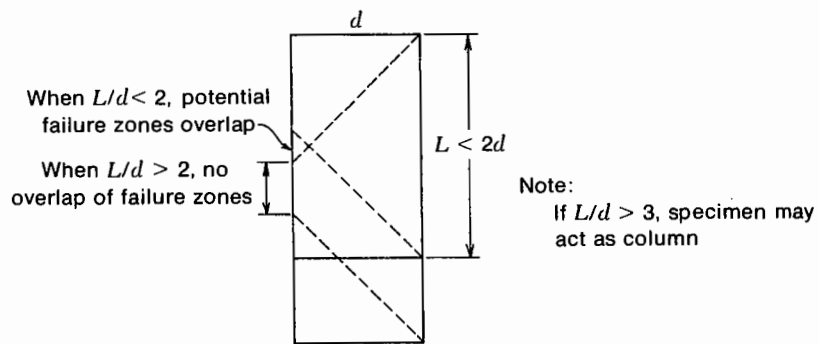


Figure 14-2

L/d ratios for any soil compression test (unconfined, triaxial, or other).

The length-diameter ratio of the test specimens should be long enough to avoid interference of potential 45° failure planes of Fig. 14-2 and short enough that we do not obtain a "column" failure. The L/d ratio to satisfy this criteria is

$$2 < L/d < 3$$

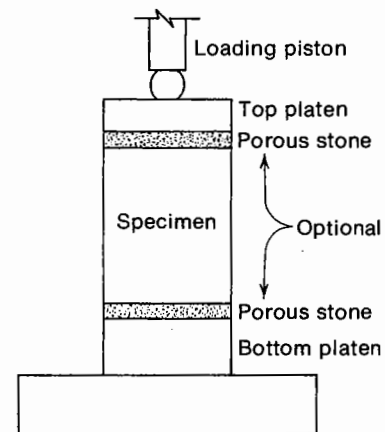


Figure 14-3

Schematic of an unconfined compression test.

Figure 14-3 gives the schematic details of an unconfined compression test. In the test an adequate length of soil sample is placed between two platens (devices to transfer load to soil), with optional porous stones inserted as shown. An axial load is applied and as the sample incrementally deforms, corresponding loads are obtained. For a considerable amount of field testing, portable compression devices are carried to the field, the sample tested to failure, and the "failure" load and deformation at failure are recorded. This data is used to compute the corrected area A' and the unconfined compression strength q_u without plotting a stress-strain curve. The q_u obtained in this manner is not likely to differ significantly from the more precise method outlined for this experiment.

Procedure *This Is a Group Project*

1. Each group will prepare two tube samples with an L/d ratio of between 2 and 3 (or use the laboratory samples furnished by the instructor).¹

¹Excellent test samples can be made using the Harvard miniature compaction apparatus. By varying the water content among laboratory groups, a study of shear strength vs. water content and/or density can be made.

2. Place the samples in moist containers or leave them in the moisture room to prevent their drying while awaiting a turn at the compression machine. Compute the deformation corresponding to 20 percent strain for your samples while waiting for the machine so that you will know about when to terminate the test if the specimen load does not peak before this amount of strain occurs.

Compute the density of your samples and weigh two moisture cups so that you can determine the water content of your samples after testing them.

3. Carefully align the specimen in the compression machine. If the ends are not exactly perpendicular to the specimen axis, the early part of the stress-strain curve will be flat (until the full sample area contributes to stress resistance, strains will be too large for the nominal stresses computed).

Set the load device to zero load (either load dial for load ring or DVM if an electronic load cell is being used) and set the deformation dial to zero. At this time a very small load should be on the sample (order of 1 dial unit of a load gage or perhaps 0.5 kg for a load cell).

Turn on machine and take load- and deformation dial readings as follows (for a 0.01 mm/div dial gage):

10, 25, 50, 75, 100, and every 50 to 100 dial divisions thereafter, until one of the following:

- a. Load decreases on sample significantly
- b. Load holds constant for 4 readings
- c. Deformation is significantly past 20 percent strain

Use the data sheet for this experiment—do not recopy the data from scratch paper (refer to Fig. 14-4). Use as many data sheets as necessary for each test. Determine the water content of each sample.

4. Compute the unit strain, the corrected area, and the unit stress, for enough of the readings (say 8 to 10 well-spaced points) to define the stress-strain curve adequately. Plot the results on a sheet of graph paper from your data sheet section (both curves on same sheet); show q_u as the peak stress of each test and show the average value of q_u for both tests. Be sure to plot strain as the abscissa.
5.
 - a. Draw a Mohr's circle using the average q_u and show the soil cohesion. *If this is not drawn with a compass the report is unacceptable.*
 - b. Compute the cohesion of the soil using the Mohr's circle of (a) and assuming the soil has an angle of internal friction, $\phi = 10^\circ$. Discuss if q_u is conservative or "unconservative" under these conditions.
 - c. Compute the secant modulus of elasticity of the soil for 0.25, 0.5 and 0.75 q_u ; also compute the *initial tangent modulus*. Tabulate these results in your "Conclusions" and discuss their significance. Which value of stress-strain modulus do you recommend for this soil?
 - d. Comment on the density and water-content effects vs. q_u .

UNCONFINED COMPRESSION TEST Type of Sample (Undisturbed, Remolded)

Data Sheet 17

Project Unconfined Compression Test Job No. ~

Location of Project Bradley University Soil Lab

Description of Soil Brown, silty clay $w_L = 37.3\%$ $w_p = 19.8\%$

Tested By SEB & KM. Date of Testing 12/7/76

Sample Data

Diam. 45.5 mm Area A_0 16.26 cm² Ht., L_0 108 mm

Vol. 175.61 cm³ Wt. 379.5 g Wet unit wt. 2.16 g/cm³

Water content, $w\%$ 14.5 Dry unit wt. 1.89 g/cm³ LRC 0.34 kg/div

Deformation dial reading ($\times 10^{-2}$)	Load dial (units)	Sample deformation ΔL , (mm)	Unit strain $\Delta L/L_0$ ($\times 10^{-2}$)	Area CF $1 - \epsilon$	Corrected area, A' , (cm ²)	Total load on sample (col. 2 \times LRC)	Sample stress, kPa
1	2	3	4	5	6	7	8
Data somewhat edited -- plotted on Fig. 14-5 Test #1							
0	0	0	0	0	16.26	-	-
50	8	0.50	0.50	0.9950	16.34	2.72	16.3
100	15	1.00	0.90	0.9910	16.41	5.10	30.5
200	20	2.00	1.85	0.9815	16.57	6.80	40.2
400	27	4.00	3.70	0.9630	16.88	9.18	53.3
500	30	5.00	4.63	0.9537	17.05	10.20	58.7
600	33	6.00	5.56	0.9444	17.22	11.22	63.9
750	38	7.50	6.94	0.9306	17.47	12.92	72.5
1,000	44	10.00	9.26	0.9074	17.92	14.96	81.9
1,200	49	12.00	11.11	0.8889	18.29	16.66	89.3
1,400	54	14.00	12.96	0.8704	18.68	18.36	96.4
1,500	56	15.00	13.89	0.8611	18.88	19.04	98.90
1,600	58	16.00	14.81	0.8519	19.09	19.72	101.30
1,700	57	17.00	15.74	0.8426	19.30	19.38	98.48

Note: Insert units in column headings as necessary.

Unconfined compressive strength $q_u = 101.5 \text{ kPa}$ Cohesion $= q_u/2 = 50.8 \text{ kPa}$

$A_v. q_u = 91.6 \text{ kPa}$ (2 tests)

Figure 14-4

Data from an unconfined compression test.

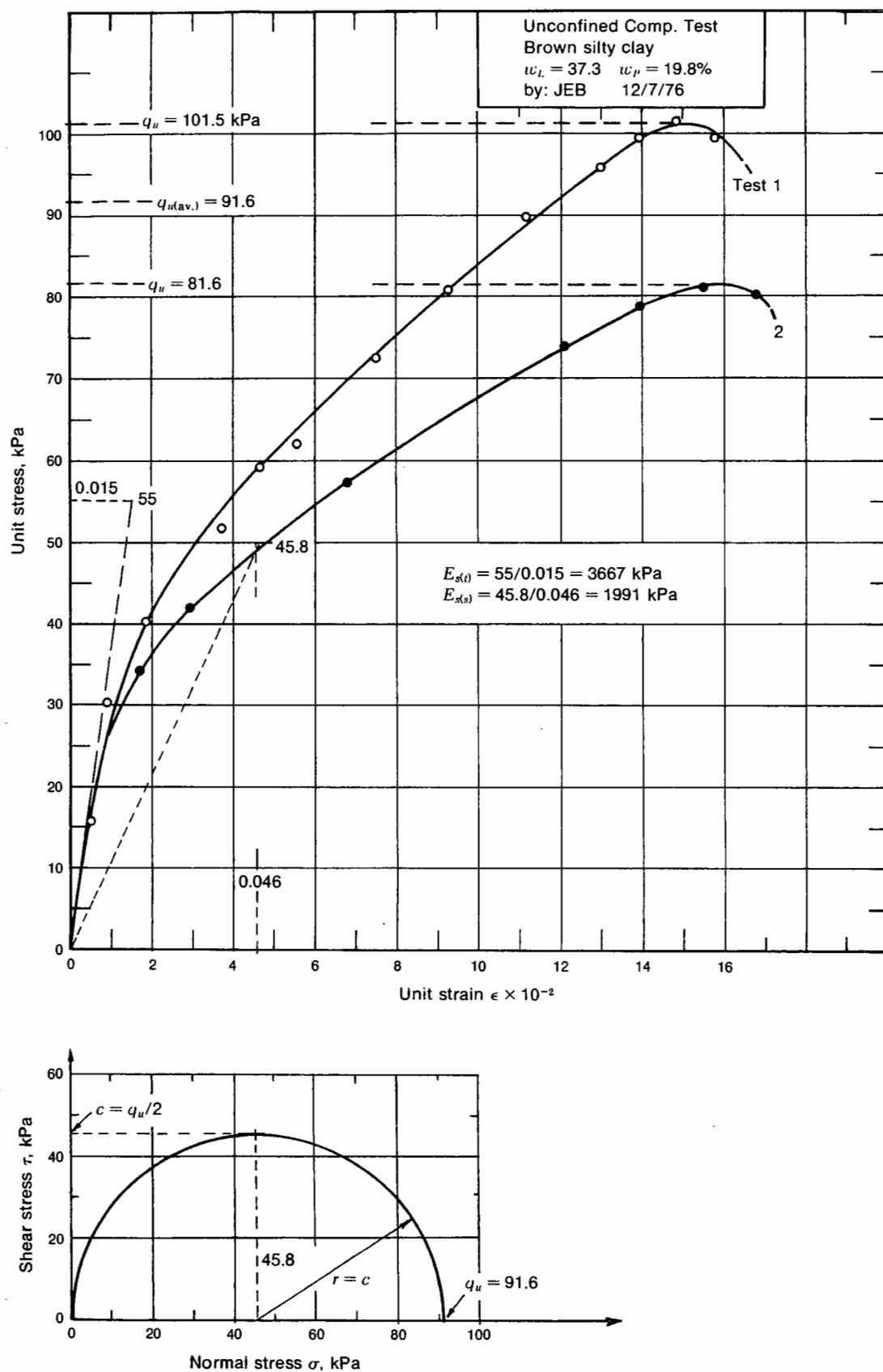


Figure 14-5

Plot of unconfined compression test data to obtain a stress-strain modulus as well as the best value of q_u . Also shown is a Mohr's circle for an unconfined compression test.

Experiment No. 15

TRIAXIAL TEST—WITHOUT PORE-PRESSURE MEASUREMENTS

References

AASHTO T234-70

ASTM D2850-70

ASTM (1949-50), Triaxial Testing of Soils and Bituminous Mixtures, *STP No. 106*.

ASCE (1960), Research Conference on Cohesive Soils, Boulder, Colorado, *Proceedings*.

Silver, M. L., and T. K. Park (1975), Testing Procedure Effects on Dynamic Soil Behavior, *J. Geotechnical Div., ASCE, GT 10*, October, pp. 1061-1083.

Objective

To introduce the student to the basic procedure for determining the soil parameters, ϕ and c , of a soil. The methods for both cohesionless and cohesive soils are included.

Equipment

Compression machine (strain-controlled)

Triaxial cell (refer to Figs. 15-1 to 15-3)

Specimen mold, rubber membrane, membrane stretcher, rubber binding strips, and porous stones

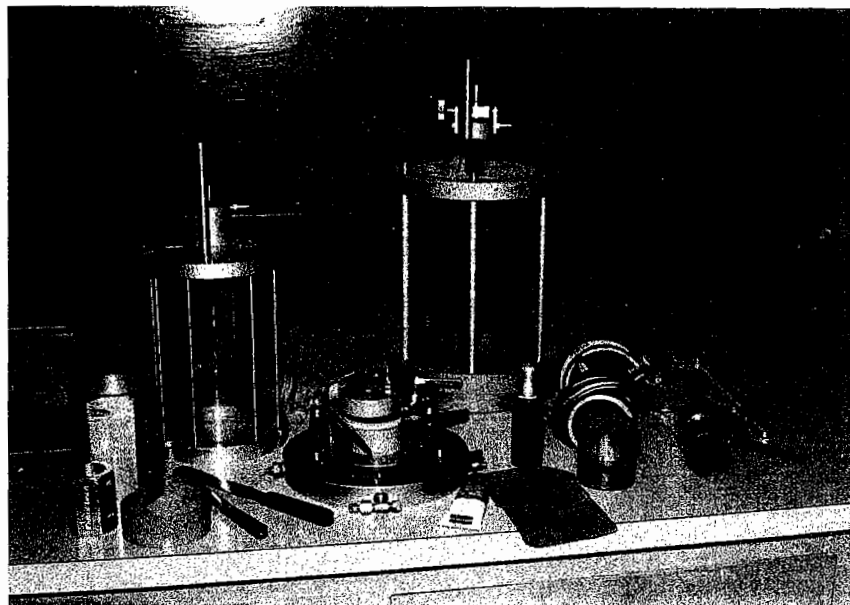
Vacuum pump and air-pressure source

Calipers

Specimen trimmer for cohesive tube samples (optional)

Figure 15-1

Triaxial cells and peripheral equipment. Two types of triaxial cells shown: quick-clamp type, which is separated, is preferred for most work but maximum cell pressure may be limited to about 5 kg/cm². Left side shows two sizes of split sleeves used to square ends of cohesive samples. Right side shows two membrane stretchers for either cohesive or cohesionless samples. Miscellaneous equipment such as spatulas, sample compactor, and trimming saw also in view.



General Discussion

Read the "General Discussion" of Experiments No. 14 and No. 16.

From an inspection of the triaxial apparatus (see also Figs. 15-1 and 15-2), the

student should be able to conclude that any soil pore-fluid state, from a negative (or vacuum) state to a fully saturated state with an excess pore-fluid pressure, can be obtained with this equipment. *Drained* or *undrained* conditions can be investigated. For a *drained* test, as the load is applied to the soil specimen, one can allow the pore fluid to escape by opening the appropriate valve. An *undrained* test can be performed by closing the soil system to the atmosphere so that no pore fluid can escape during the test. To most persons, the term *drained test* means that the proper valve is opened so that the soil pore fluid can escape. Strictly speaking, however, a drained test is one in which the proper pore-fluid drain valve is opened and the rate of stress application is so slow that, within the sample, *no excess pore pressure* exists during the test. To distinguish among the different types of tests which can be performed by opening or closing the pore-fluid exit valve, three tests will be described:

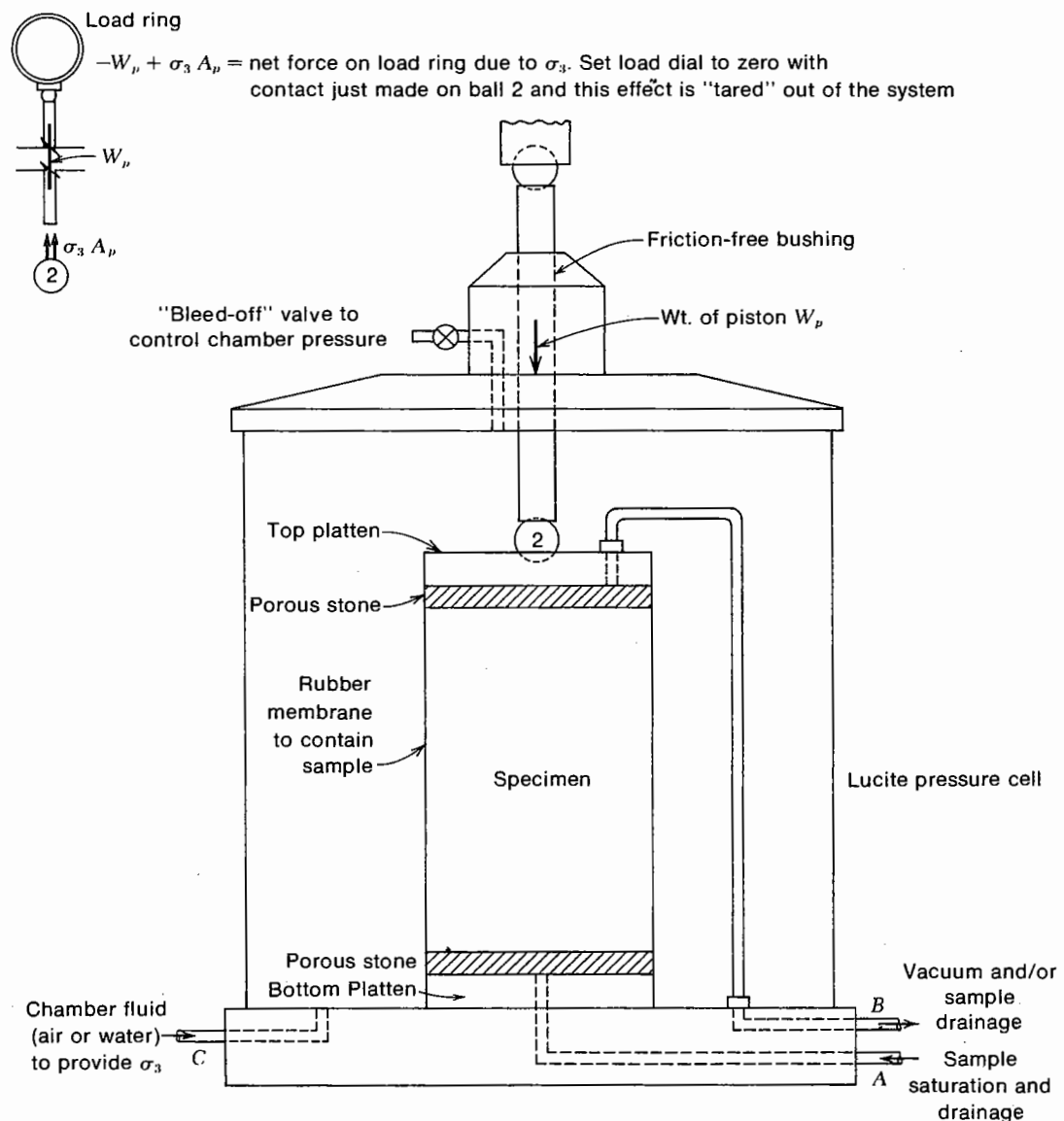


Figure 15-2

Line details of the triaxial cell. Note that shutoff valves will be located on tubes A, B, and C. If an electronic pore pressure transducer is used, it would be on a T connection on tube A.

1. *Undrained test*—also called an *unconsolidated-undrained* test or U test. This test is performed with the *drain valve closed* for all phases of the test and before the sample has a chance to consolidate (if $S < 100$ percent). The test is commenced immediately after the cell pressure is stabilized. For fine-grained soils—and particularly if $S \neq 100$ percent—it should make little difference whether the drainage valve is open or closed during the test. This test cannot be used for a cohesionless soil of $S = 100$ percent.
2. *Consolidated-undrained* test—also termed a consolidated-quick or CU test. This test takes place after the sample has consolidated with the drain valve open under some confining cell pressure. Some type of sample monitoring device is needed to determine when the sample volume change has halted (or the pore fluid ceases to drain), indicating that consolidation is complete. As consolidation takes a considerable time for clay samples, special techniques are used such as placing a sand core in the specimen or using filter paper strips or cotton threads spaced around the sample perimeter adjacent to the rubber membrane to speed drainage.

When consolidation is complete, the *drain valves are closed* and the deviator stress is applied to sample failure. This latter phase of the test is identical to the U test.

3. *Consolidated-drained* test—also called consolidated slow test or CD test. In this test the *drain valve is opened*, the cell pressure applied, sample is monitored for complete consolidation, and then, with the drain valve still open, the deviator stress is applied. The deviator stress must be applied at such a slow rate that pore pressures, which are always developed during shear, are small enough that the soil parameters are not affected.

Because of the close monitoring and considerable length of time involved, this test can be economically justified only for large projects. While this test produces the true soil parameters (ϕ and c), on most projects the apparent parameters obtained using either undrained or CU tests are more realistic—depending on how rapidly the in situ loading, e.g., a fill or footing load, is applied.

TEST RESULTS

1. For any dry soil (both cohesionless and cohesive), about the same soil parameters would be obtained from any of the 3 tests (U, CU, or CD).
2. For a saturated or partially saturated cohesionless soil, the CD test will yield about the same ϕ angle as for a dry soil unless the material is very fine-grained (low coefficient of permeability) and/or the test is performed at an extremely rapid rate of strain.
3. Any saturated cohesive soil will yield $\phi = 0$ in a U test to possibly a small ϕ angle in a CU test.
4. For any saturated cohesive soil, the results are highly dependent on which of the three tests is used. That is, the soil parameters will range from:

$$\begin{array}{ccc}
 \left. \begin{array}{l} \phi \cong 0 \\ \text{and} \\ c = \text{a value} \end{array} \right\} \text{U test} & \begin{array}{l} \text{to } \phi = \text{true value} \\ \text{and } c \cong 0 \end{array} & \left. \right\} \text{CD test}
 \end{array}$$

for the same soil. Results will depend also on whether the soil is normally consolidated, overconsolidated, or is a remolded sample. The data presentation should indicate these factors and the type of test used to obtain the results. The soil parameters also depend upon making allowance for pore pressures (see Experiment No. 16) in solving Coulomb's equation for shear strength; i.e., we may be able to measure the pore pressure in a CU test and correct the stresses to obtain the true soil parameters.

5. For any partially saturated cohesive soil, the results depend on both the degree of saturation and the *type of drained test* performed. The results from an undrained test will be highly dependent on the degree of saturation of the soil sample, ranging from

$$\begin{array}{ll} \phi = 0 & \text{for } S = 100 \text{ percent} \\ \text{to } \phi = \text{true value} & \text{for } S = 0 \end{array}$$

DATA OBTAINED

The triaxial tests yield data to plot a Mohr's stress circle using the cell pressure as σ_3 and the corresponding major principal stress σ_1 at sample failure. By plotting two to four Mohr's circles using test data based on different initial cell pressures σ_3 for each test and on samples of soil with approximately the same density and water content, an approximate tangent (or stress envelope) to the circles can be established. The slope of this tangent is taken as the angle of internal friction, ϕ , of the soil, and the intercept of the stress envelope on the y axis is taken as the cohesion c in Coulomb's equation:

$$\tau = c + \sigma_n \tan \phi \quad (15-1)$$

For cohesionless soils, the cohesion intercept should be small unless the soil is so damp that surface tension may be appreciable. There is generally a small intercept (usually termed *apparent cohesion*) for dry cohesionless soils due to the rubber membrane used to surround the sample and due to test errors, but values of 7 to 14 kPa are generally neglected. For apparent cohesion values larger than these, the possible reason for the occurrence should be investigated. For a single test on a cohesionless soil ($c = 0$), the angle of internal friction can be computed from the geometry of a Mohr's circle as

$$\sin \phi = \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3} \quad (15-2)$$

In order to obtain the major principal stress σ_1 , it is necessary to know the confining chamber pressure since this pressure acts both on top of the sample and on the sides. Thus, with a chamber pressure of 70 kPa, the value of σ_3 is 70 kPa, and at this instant, before any loading, the value of σ_1 is also 70 kPa. With the application of external load via the loading piston, the major principal stress increases as

$$\sigma_1 = \sigma_3 + \frac{P}{A'} \quad (15-3)$$

where P = the instantaneous machine load on the specimen at some time after the test is started

A' = the corrected area of the specimen, as in Experiment No. 14

$$A' = \frac{A_0}{1 - \epsilon} \quad (14-4)$$

The load piston which applies additional load to the specimen is carefully machined and fitted with ball-joint connections to provide an airtight, friction-free (nearly), axial load application to the sample inside the triaxial cell.

The value of P/A' of Eq. (15-3) is often called the *deviator stress*, a term used previously in this discussion. It is evident that load and deformation readings must be obtained for this test as for the unconfined compression test and that a

plot of either deviator stress P/A' or principal stress σ_1 versus unit strain must be made. The peak value of stress (or the stress value occurring at 20 percent strain if it occurs first) is taken from the stress-strain plot as the major principal stress, or the maximum deviator stress, for use in plotting the Mohr's circle.

If a calibrated burette is attached to the sample drain outlet and if the sample is saturated, changes in sample volume can be observed under the different deviator loads. Completion of consolidation for consolidated tests can also be determined using the burette connection since no volume change or change in water level in the burette should occur after the sample consolidation is complete and prior to application of a deviator stress. If the sample is not saturated, volume changes occurring in the sample may not accurately be indicated by water-level changes in the burette since the water may not drain but may instead remain in the sample, resulting in a slight change in water content.¹

Special attachments are available, or can be constructed, to measure the pore pressure in the sample and changes in pore pressure under load. This method of triaxial testing is considered in Experiment No. 16.

Test-specimen diameters are standardized so that the load platens and porous stones required for a test can be installed in cells of more than one size, giving the pressure cells considerable versatility. Common diameters are

36, 47, 50, 63.5, 71, 76, and 100 mm (nominal dimensions)

Rubber membranes are required to confine the specimen, and these are available in the above sizes.² The length/diameter ratio should be

$$2 < L/d < 3$$

as for the unconfined compression test.

The strain rate can be taken at

0.5 to 1.25 mm/min

except for the CD test where it may be necessary to use a strain rate on the order of 0.0025 mm/min or less, depending on the size of the sample and the grain size.

SPECIAL TRIAXIAL TESTS

1. The triaxial cell can be used to build laboratory specimens using *isotropic* or *anisotropic* consolidation.

a. Isotropic consolidation—build soil sample of desired size in membrane. Apply the cell pressure σ_3 and place the sample drainage tube in a container of water so it can drain without desiccation (or attach to a volumetric device so consolidation can be monitored). Use filter-paper strips or threads between membrane and soil to speed consolidation. Use filter paper between porous stones and soil to reduce stone contamination.

b. Anisotropic consolidation—build sample as for isotropic consolidation. Construct a dead-load frame and apply a vertical load in conjunction with the cell pressure σ_3 to develop the desired stress ratio of

$$K = \frac{\sigma_h}{\sigma_v}$$

¹See Experiment No. 16 for a method of determining volume change for partially saturated soil samples.

²A very economical solution for the 36- and 38-mm-diameter cohesive soil samples is to use condoms (any drug store) for the membrane. Cut the end off with a pair of scissors, put on lower platen, place sample, and then simply roll the membrane on without the use of a membrane stretcher; test setup takes about half the time of using membrane stretcher.

2. Soil tests can be performed using the triaxial cell for conditions of

$$\sigma_v < \sigma_3 \quad (\text{normally vertical stress is larger})$$

but will require one or both of the following:

- a. Cell modification to isolate the vertical stress from the lateral stress (i.e., for small-diameter samples remachine the load piston/cap assembly to the size of the sample so that no σ_3 acts on top of the sample.
 - b. Perform test using a dead-load frame and incremental dead loads; with some dead-load stress and cell pressure, remove dead loads (reducing vertical stress) in increments until failure occurs. Dead load may consist of a large container of water with a drain valve, swung from a load cell on the dead-load hanger.
3. Dynamic cyclic tests may be performed. These tests require special equipment not likely to be found in most soil-testing laboratories. Steps in a dynamic or cyclic test include (see also Silver and Park, 1975):
- a. Build sample to the desired density.
 - b. Apply a specified strain ϵ of, say, 2 percent, 4 percent, etc.
 - c. Apply a cyclic strain (often $\pm\epsilon$ of step 2) at some frequency (Hz)
 - d. Using electronic data acquisition equipment, record load-cell stress at the corresponding strain. Until the sample deforms excessively, the cyclic stress-strain curve (obtained as direct electronic recording using some type of pen recorder) is somewhat as in Fig. 15-3.
 - e. Apply a new strain increment ϵ , and repeat steps c and d until the sample "fails." Figure 15-3 illustrates typical (and qualitative) results from two strain increments at low strain levels. This typical curve is considerably distorted at higher strain levels.

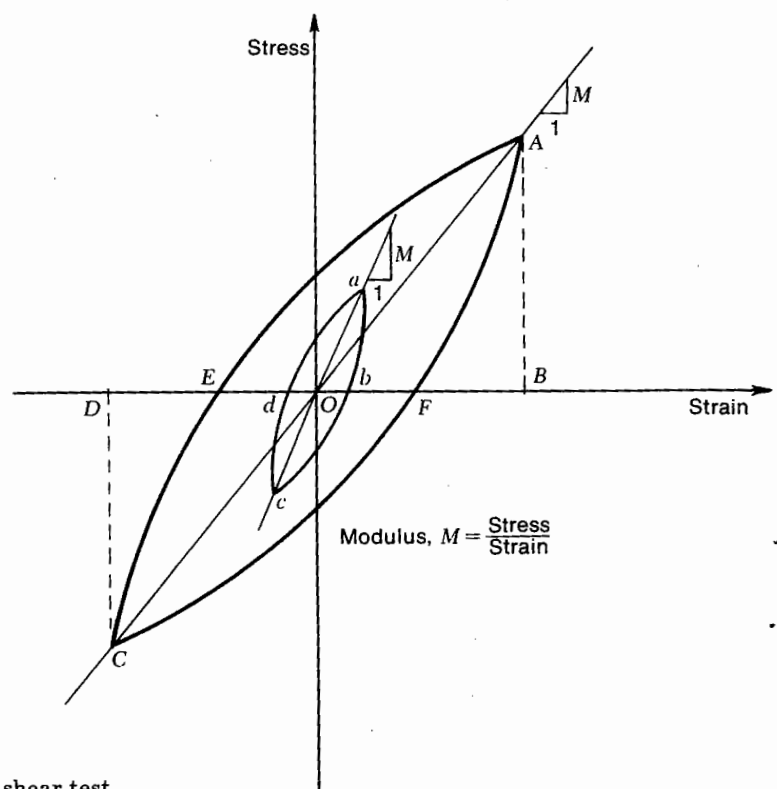


Figure 15-3
Qualitative stress-strain plot of cyclic shear test.

Cyclic tests may be performed using either compressive (axial) strain or shear strain. If axial strain is used, the *dynamic* modulus of deformation (M of Fig. 15-3) is the conventional dynamic modulus of elasticity E_{sd} . If sheer strain is used, the modulus is the dynamic shear modulus of elasticity G_d . A dynamic damping factor can be computed for each strain increment, e.g., as:

$$\beta = \frac{1}{2\pi} \frac{\text{area of loop } AFCE}{\text{area of triangles } OAB + ODC} \quad (15-4)$$

of Fig. 15-3. The peak-to-peak strain is (from Fig. 15-3, test 2)

$$\epsilon_{pp} = BD$$

with an amplitude of $BD/2 = OB$.

RESILIENT MODULUS

Sometimes a triaxial test is taken up to some stress level, say, 0.25 to 0.50 of the estimated ultimate value, the deviator load reduced to zero, then reapplied. This sequence may be repeated three to five times; on the last cycle the deviator load is then continued to sample failure (see Fig. 15-8, test 1). There is some body of opinion that the *initial tangent modulus* (which is somewhat larger than on the initial cycles with the increase considered due to "strain hardening") is a better estimate of the modulus of elasticity than otherwise obtained.

In performing this test, care is necessary in unloading the sample of the deviator stress to obtain the new strain at a deviator stress of zero since the sample undergoes permanent (plastic) deformation and very little elastic recovery is obtained as shown in actual (but edited) data of Fig. 15-7. This can be accomplished by:

1. Unloading the sample (deviator stress = 0) immediately (the unload branch of the stress-strain curve is of academic interest only).
2. Carefully reapplying a very small load increment and then reading the deformation dial. The difference between the zero reading and this reading will be the plastic deformation of the sample at the end of that load cycle.

Procedure

A. COHESIONLESS SOIL—LABORATORY WORK

1. Fasten the base platen to the base of the cell.¹ Attach a rubber membrane of the proper diameter to the base platen (refer to Figs. 15-2 and 15-4) using rubber strips or bands. To provide a more impervious joint, the base platen may be lightly coated with silicone grease prior to attaching the membrane; this will increase the seal between the membrane and the platen.

Some persons measure the wall thickness of the rubber membrane so that an area adjustment can be made for the initial area of the sample, but for ordinary computations it is the author's opinion that this introduces a fictitious precision into the results.

Place a porous stone on the base of the platen. If the sample is damp or is to be saturated, the porous stone should be soaked² prior to use.

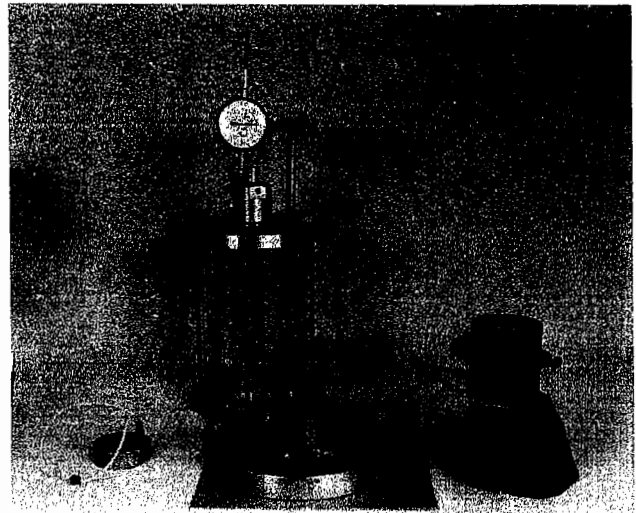
¹For sand, the platen should be at least 63 mm in diameter. If gravel is present, the diameter should be about six times the largest grain size.

²See step 1 of Procedure A, Experiment No. 16.

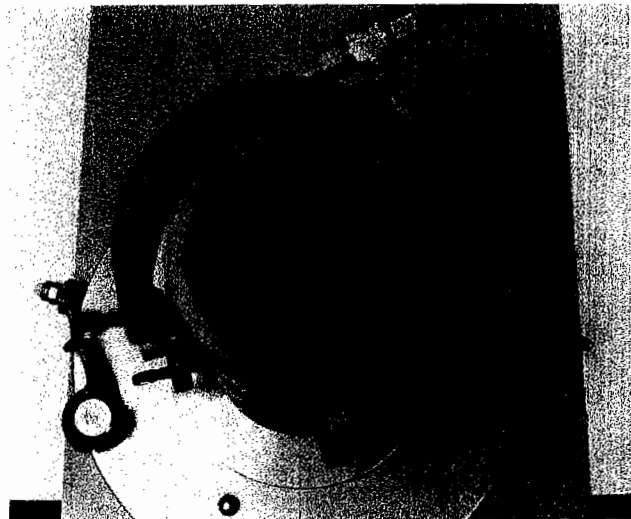
Figure 15-4

Details of a cohesionless triaxial sample construction/test.

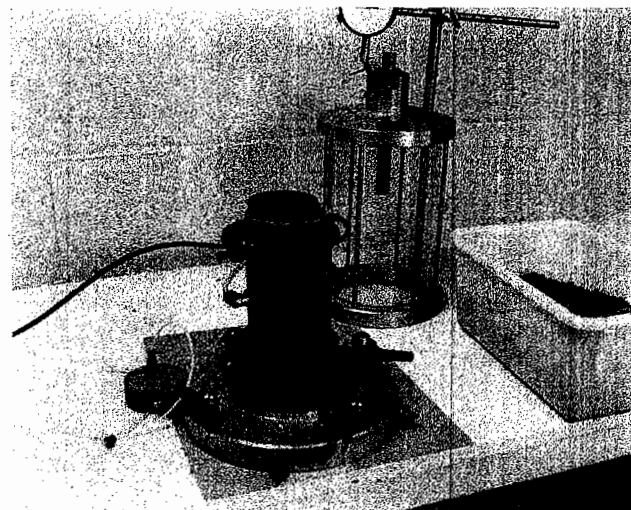
(a) Equipment needed includes cell, two porous stones, membrane stretcher, membrane, and top and bottom platens of appropriate size.



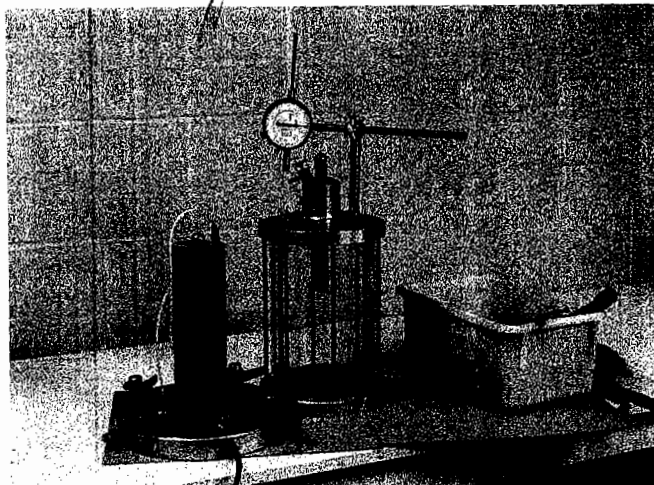
(b) Membrane, inside membrane stretcher, placed over bottom platen and bottom porous stone.



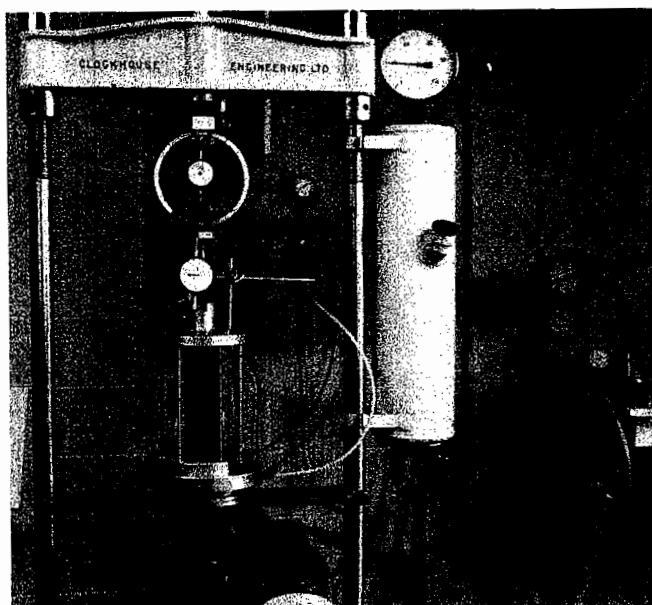
(c) Sample nearly constructed of damp sand (in this case) and level with top of mold/membrane stretcher. Porous stone shown goes on next.



(d) Sample built, membrane stretcher removed, and load piston ball in position ready to attach cell cover.



(e) Cell in position in one type of compression machine: Upper dial is for load cell, lower dial is for sample deformation. Tank at right is a cell-fluid reservoir that allows a takeoff for air pressure which author generally uses. On right is shown dual system of pressure reduction (pressure gages for both fps and metric values) which accurately controls cell pressure and maintains reasonable cost since even with filtering air contains water and is corrosive to expensive pressure-control equipment.



2. Weigh a container of dry sand so that the sample density can be obtained and approximately duplicated for succeeding tests.

If it is necessary to produce a sample of given density, the dry sand must be mixed with a known percent of water so that a wet sample is used. It is nearly impossible to produce a required density—especially higher densities—using a dry sample.

3. Place a specimen mold around the rubber membrane and fold the top portion of the membrane down over the mold. Do not fold, however, if the mold is so large that the membrane will be torn in the process. If they are available, membrane stretchers may be used for forming the sample. When a split-barrel type which can be attached to a vacuum is used, it may be necessary to apply silicone grease along the split to effect a more efficient vacuum seal. For the split stretcher and other types of stretchers using a vacuum, it may be also necessary to use threads or small strips of filter paper between the membrane and stretcher wall so that the vacuum will be effective the full height of the membrane stretcher.

4. Carefully place the sand in the membrane using a tamper to maintain the sample shape and density. When producing a sample of required density, to maintain the density constant through the sample height, it may be desirable to divide the height into equal increments, compute the quantity of (wet) soil for each height, and place that portion in each sample segment. This is not an easy process and the sample may have to be done over several times before the desired density is obtained.
5. Place a porous stone (damp for wet soil) and then the top platen on the sample. It may also be necessary to coat the outer rim of the platen with silicone grease to obtain a more leakproof seal. Roll the membrane off the mold onto the top platen and seal with rubber strips or bands. Take a small level and level the top platen.
6. Attach the tube from the top platen to the vacuum outlet (see Fig. 15-2) in the base of the cell and apply a vacuum of 200 to 250 mm of mercury to the sample (outlet B is closed).
7. Now remove the specimen mold and examine the membrane for holes and obvious leaks. If any are found, the sample must be remade using a new membrane.
8. Obtain four height measurements approximately 90° apart and use the average value for the initial specimen height L_0 . Take two diameter readings 90° apart at the top, at midheight, and at the base using a pair of calipers. Take these measurements to the nearest 1 mm. Compute the average diameter of the specimen at each height location and then compute a final average specimen diameter as

$$d_{av} = \frac{d_t + 2d_m + d_b}{4}$$

where d_t is the average diameter based on the two top measurements, etc. Compute the corresponding value of initial sample area A_0 using the average diameter just computed.

9. Place the lucite cylinder on the cell base,¹ being sure the base is free of soil grains so that an airtight seal can be obtained. Place the cell in the compression machine and just barely make load contact of the loading piston and the loading bar (or crosshead) of the compression machine.
10. Apply a predetermined lateral pressure σ_3 to the cell (preferably in even multiples of N/m² for computational ease) using compressed air and simultaneously reduce the vacuum on the interior of the sample to zero. Some persons prefer using a liquid chamber fluid (often water with a rust inhibitor added or glycerine) rather than compressed air. The liquid has the advantages of providing a more uniform pressure and of being more viscous, and hence, not as sensitive to membrane leaks. For student laboratories it has the disadvantages of requiring more laboratory time (for siphoning out the cell at the end of a test) and of being more messy—especially if any leakage occurs. Now with vacuum shut off at A, open outlet B (Fig. 15-2) and check if any air is coming out under pressure which indicates a sample leak and necessity to start over.
11. If it is desired to saturate the sample, open the appropriate valves and observe the sample until it is saturated.² A slight vacuum on the sample can be used to speed up the saturation process. Alternatively, a positive pressure (termed *back pressure*) of a value less than σ_3 , say, $\sigma_3/2$, can be applied to the

¹Note that it is possible to supply the lateral pressure of σ_3 by the vacuum on dry, cohesionless samples. If a vacuum is used, it is not necessary to use a cell cover. The maximum reliable σ_3 will not be over 90 to 96 kPa, depending on the efficiency of the vacuum pump.

²See Experiment No. 16 for refinements in saturating a cohesionless soil sample.

saturation reservoir. If you use too large back pressure, the sample will blow up. The load piston should be snugly fitted to the specimen cap so the sample does not expand under the back pressure, although this should not occur for any back pressure less than the chamber pressure.

12. When the chamber pressure was applied, it could be seen that the load dial (or DVM) recorded an upward load which is the *difference* between the weight of the load piston and the upward chamber pressure on the piston base. Carefully raise the cell until the load (or DVM) dial just records a load, indicating that recontact of the piston and soil sample is made. Now zero the load dial (or DVM) to tare the system approximately. If this is done, the load dial (or DVM) will be recording the corrected deviator load for more convenient computations.
13. Attach a deformation dial (reading to 0.01 mm/div) to the machine so that the sample deformation can be obtained. Set the dial gage to zero; then manually compress and release the dial plunger several times and observe the zero reading. Readjust the gage to zero if necessary. Note that a LDVT may be used instead of a dial gage but is not recommended for student laboratories. Check deformation dial gage, load gage, and cell pressure gage for final correct settings.
14. Set the compression machine to the desired strain rate (generally between 0.5 and 1.25 mm/min) as specified by instructor.
15. Turn on the compression machine and take simultaneous load- and deformation readings using data sheets from your data sheet section. Readings may be taken at

5, 15, 25, 50, and every 50 to 100 dial divisions until

- a. Load peaks and then falls off
- b. Somewhat past 20 percent strain
- c. Load holds constant for 3 or 4 successive readings

Compute the corresponding stress (kPa) and strain for sufficient readings to define the stress-strain curve (8-12 points).

Be sure to monitor the chamber pressure gage and do not let the chamber pressure vary by more than 0.05 kg/cm² (5 kPa). It takes only slight pressure changes to alter considerably the deviator stress which defines "failure."

16. After the sample fails, shut off and/or reverse the compression machine; release the chamber pressure and remove the sample load.
17. Prepare a new specimen to the same approximate density (within 0.2 to 0.5 kN/m³) and make two additional tests. This completes the laboratory work; refer to Procedure C to complete report.

B. COHESIVE SOIL—LABORATORY WORK

1. Prepare two or three tube samples of adequate L/d ratio. Alternatively, compact three or four samples at some constant water content and compaction effort using the Harvard miniature compaction apparatus. This latter procedure will yield samples of 33 mm nominal diameter and of adequate L/d ratio.¹
2. Obtain four height measurements approximately 90° apart and use the average value for the initial specimen height L_0 . Take two diameter readings 90° apart at the top, at midheight, and at the base using a pair of calipers. Take these measurements to the nearest 1 mm. Compute the average diame-

¹It is also possible to compact samples in the standard compaction mold (including filling the collar) and then use a sample trimmer to reduce the size of the sample to one that is convenient for this test.

ter of the sample at each height location and then compute a final average specimen diameter as

$$d_{av} = \frac{d_t + 2d_m + d_b}{4}$$

where d_t is the average diameter based on the two top measurements, etc. Compute the corresponding value of initial sample area A_0 using d_{av} from the above equation.

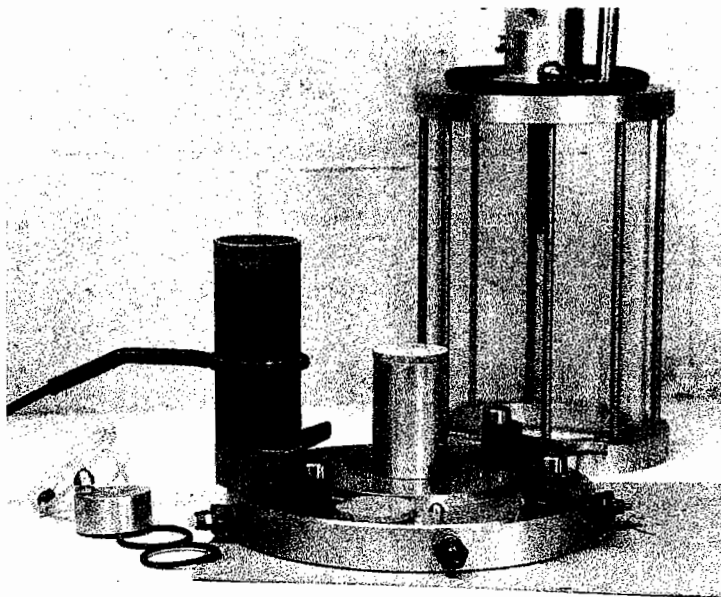
3. Take the correct-size (exact to say +6 mm larger diam.) membrane stretcher and membrane and fit the membrane smoothly into the stretcher, folding the ends of the membrane over the ends of the stretcher (Fig. 15-5). Attach the tube on the membrane stretcher to a vacuum source and apply a vacuum. If there are no leaks, the membrane will form a smooth cover on the inside of the stretcher under the vacuum. It may be necessary to place some small threads or filter-paper strips between the membrane and the walls of the stretcher to make the vacuum effective when stretching the membrane tight prior to the next step.
4. Depending on the sample, either lightly lubricate the membrane with petroleum jelly, powder it with teflon powder (an inert substance), or as a last resort, moisten it with water for ease of insertion of the sample into the membrane.
5. Insert the sample into the membrane and attach the lower platen and porous stone using rubber bands or strips to seal the membrane. A more airtight seal may be obtained if the platen perimeter is lightly greased with silicone grease. If possible, attach the upper platen with its porous stone at this time. Be sure to use damp or saturated porous stones on the platens, depending on the sample condition, as a dry stone will absorb water from the ends of the sample.
6. Remove the sample from the membrane stretcher and attach the lower platen to the base of the triaxial cell. Also attach the upper platen if this has not already been done. Use extreme care not to damage the soil specimen.
7. Connect the top-platen tube to the vacuum line, *but do not apply a vacuum*. If a consolidated-undrained test (on a saturated sample) is to be run, flood the lines into and out of (i.e., the vacuum line) the sample. Then close the valve of the line into the sample, leaving the vacuum line open for drainage. Some ingenuity is required to ensure that the lines, top porous stone, and top platen orifice are saturated. For unsaturated samples, line flooding is not necessary and may actually be harmful.
8. Place the lucite cover on the cell and place the cell in the compression machine. Bring the load bar into contact with the load piston until a load just flickers on the load dial.
9. Apply a predetermined chamber pressure (preferably compressed air) for the lateral pressure σ_3 . For the consolidated-undrained test on a saturated sample, observe the flow of fluid out of the drain line and, when the flow is zero, proceed with steps 11 and 12 following. The drain line may be connected to a calibrated burette so that when the water level remains constant, the consolidation can be assumed to be complete. This type of connection will also yield a volume change during consolidation. If the sample is not saturated, it becomes an exercise in judgment¹ as to when the consolidation is complete.
10. If it is desired (on initially unsaturated samples) to saturate the sample, open the vacuum line to the atmosphere and open the inlet valve to the sample from the saturation reservoir. Do not apply a vacuum to the sample to speed the saturation process as this will remove water already in the sample on the

¹Refer to Experiment No. 16; Alternatively, attach the deformation dial and monitor when vertical movement has halted as a indication of 100 percent consolidation.

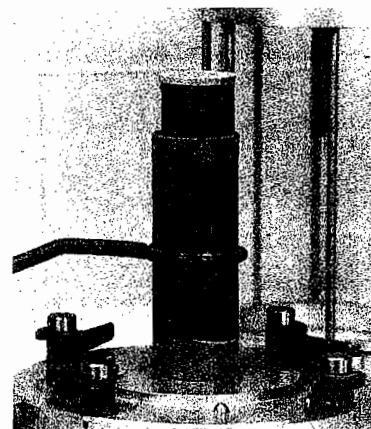
Figure 15-5

Cohesive triaxial test.

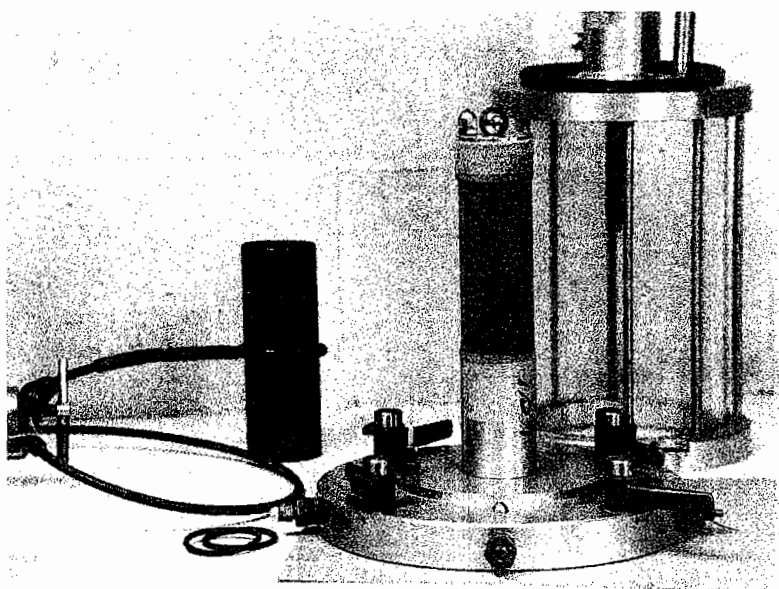
(a) Membrane has been placed on membrane stretcher with vacuum attached. In this case the membrane stretcher is 3 mm larger than sample.



(b) Membrane stretcher is placed on bottom platen (with porous stone in place) and sample is inserted without difficulty because stretcher is oversized. It is quite easy to place top porous stone and platen in place and roll the membrane off stretcher to effect sample seal.



(c) Membrane stretcher removed and sample ready to be enclosed in cell. Rest of test same as for cohesionless sample (with adjustment for U, CU, or CD type test).



vacuum side faster than water will flow into the sample from the source because of the low permeability of the sample. It will take a long time to saturate even small cohesive samples. It may be possible to put a positive pressure into the saturation reservoir to speed the process as long as the pressure is well below the chamber pressure.

11. If the sample is already saturated (and/or consolidated) and it is desired to measure volume changes, open the appropriate valves to fill the lines on both sides of the sample and then close the drain outlet. Open the line to the volume-change indicator (the burette). To speed the consolidation process of saturated samples when performing consolidated tests, one may place a series of small threads or strips of filter paper between the soil sample and the membrane in step 5.

From this point on, refer to steps 12 to 17 of Procedure A and all of Procedure C as the work is identical.

C. COMPUTATIONS

1. Compute the unit strain from the deformation-dial readings as

$$\epsilon = \frac{\Delta L}{L_0}$$

and fill in the appropriate column of the data sheet. Also compute the adjusted area

$$A' = \frac{A_0}{1 - \epsilon}$$

and place this in the appropriate column of the data sheet (see Figs. 15-6 and 15-7).

2. Compute the *deviator* load using the load-dial readings. If a load ring is used, the deviator load P is

$$P = DR \times \text{load-ring constant}$$

where DR is the load-dial reading in units of deflection. Put these data in the appropriate column of the data sheet.

3. Compute the deviator stress

$$\sigma = \frac{P}{A'}$$

and fill in the appropriate column of the data sheet.

4. Plot a curve of unit deviator stress vs. unit strain (on the abscissa) and obtain the stress at the peak point unless the stress at 20 percent strain occurs first (see Fig. 15-8). Show this value of deviator stress on the graph. Plot all three tests on the same graph sheet but be sure the curves are identified for the corresponding values of confining pressure σ .
5. With the maximum deviator stress from step 4, compute the major principal stress for each test as

$$\sigma_1 = \sigma_3 + \sigma$$

6. Plot Mohr's circles for the three tests on the same set of axes and fit a tangent to the circles. Measure the slope of the tangent as the angle of internal friction ϕ of the soil and show on the plot. Measure the y -axis intercept for the

cohesion of the soil and show this value on the graph. For cohesionless soils, a large value of apparent cohesion should be discussed in the report.

7. Compute ϕ for each test on cohesionless soils using Eq. (15-2). Average the values and compare with the value obtained from the Mohr's circle plots of step 6 above. Derive Eq. (15-2) on the sample computation sheets if cohesionless soils have been tested.
8. Compute

$$p = \frac{\sigma_1 + \sigma_3}{2} \quad \text{and} \quad q = \frac{\sigma_1 - \sigma_3}{2}$$

for each test and plot the points (p, q) using p on the abscissa. Connect the points with a smooth curve (called a *stress path* or *K line*). Take an average line through the curve and measure the slope as α and the q intercept (ordinate value) as a .

Compute ϕ based on

$$\sin \phi = \tan \alpha$$

and compute the cohesion as

$$c = \frac{a}{\cos \phi}$$

How do these values of ϕ and c compare with those obtained from step 6 above? What would be the practical significance of using the deviator stress at some specified strain (say, 10 percent) to obtain σ_1 to compute p and q for plotting the *K line* instead of the peak values?

9. Compute the *tangent modulus* and the *secant modulus* using the slope of the stress-strain curve at one-half peak value for the tangent modulus and the origin and one-half peak value for the secant modulus. Comment on the location along the stress-strain curve used and on any differences in values obtained.
10. Show a neat line drawing of the laboratory test setup as part of the report regardless of type of soil tested.
11. If cohesive soils have been tested, be sure to comment in the report and *show on the Mohr's circle plot* the type of test performed (that is, U test, CU test, or CD test) and whether the soil is saturated or unsaturated.
12. Using Hooke's generalized stress-strain law as

$$\epsilon = \frac{1}{E_s}(\Delta\sigma_1 - 2\mu\sigma_3)$$

where ϵ = strain at two points on the stress-strain curve sufficiently close that you may assume $E_s = \text{constant}$

$\Delta\sigma_1$ = corresponding deviator stress values

μ = Poisson's ratio

σ_3 = cell pressure for curve of interest

Obtain sufficient points along the curve to determine the strain level at which Poisson's ratio becomes either (–) or larger than 0.5 (either of which indicates a nonelastic strain level). It may be convenient to program this on a computer or programmable pocket calculator.

In your report make appropriate comments on the values of E_s and μ and the stress (and strain) level at which Poisson's ratio becomes invalid for an elastic material.

TRIAXIAL COMPRESSION TEST (Cohesive, Cohesionless)

Data Sheet 18

Project Resilient modulus test Job No. ~
 Location Bradley University Boring No. ~ Sample No. ~
 Description of Soil Blue clay $G_s = 2.72$ Depth of Sample ~
 Tested By JEB & RGL Date of Testing 1/12/77

Fill in the blanks with data and appropriate units.

Sample Data

Dimensions of test specimen: Diam. $D_0 = 3.335$ Area $A_0 = 8.735 \text{ cm}^2$ $L_0 = 7.09 \text{ cm}$

Vol. $V_0 = 62.02 \text{ cm}^3$ Water content $w = 13.7\%$ Degree of Saturation, $S = 61.0\%$

Wt. $W_0 = 119.089$

For Cohesionless soils

Initial wt. of container + sand = ~

Final wt. of container + sand = ~

Wt. of sand used in specimen, $W_s =$ ~

Specific gravity of sand $G_s =$ ~

Vol. of soil solids in test specimen $V_s =$ ~

Vol. of voids in test specimen (initial) $V_v =$ ~

Initial void ratio of test specimen $e_i =$ ~

Void ratio of sand at minimum density $e_{\max} =$ ~

Void ratio of sand at maximum density $e_{\min} =$ ~

Relative density of test specimen $D_r =$ ~

Unit wt. of test specimen (cohesive, cohesionless) $\gamma = W_0/V_0 = \frac{119.089 \times 9.807}{62.02} = 18.83 \text{ kN/m}^3$

Machine Data

Rate of loading 0.127 cm/min (insert centimeters or inches)

The following data may not be applicable if machine can be adjusted to tare these effects out of the load readings.

Cross section area of loading piston, $A_p =$ ~ cm^2

Upward load on piston $= A_p \sigma_3 =$ ~ kg

Wt. of loading piston $=$ ~ kg

Computed value of initial sample load $=$ ~ kg (If upward load is larger than weight of piston, ignore.)

Figure 15-6

Preliminary sample data for cohesive triaxial test.

TRIAXIAL COMPRESSION TEST

Data Sheet 19

Project Resil. Modulus test Job No. ~

Location of Project Bradley Univ. Boring No. ~ Sample No. ~

Description of Soil Blue Clay Depth of Sample ~

Tested By JEB & RGL Date of Testing 1/12/77

Sample Data: Area $A_0 = 8.735 \text{ cm}^2$ Length $L_0 = 2.09 \text{ cm}$ (5.68 N/d.v.)

Machine Data: Load rate = 0.127 cm/min Load ring constant LRC = 0.579 kg/div.

	Deform. dial reading ($\times 0.01$)	Load dial reading	ΔL in. (col. 2 $\times 0.01$)	Unit strain $\Delta L/L_0$	Area correction factor $1-\epsilon$	Correct. area A' (cm^2)	Deviator stress" kPa
1	2	3	4	5	6	7	8
1 st cycle	0	0	0	0	1.000	8.74	0
	25	3	0.025	0.004	0.996	8.77	19.4
	50	9	0.050	0.007	0.993	8.79	58.2
	75	16	0.075	0.011	0.989	8.83	102.9
	85	17	0.085	0.012	0.988	8.84	109.3
2 nd cyc.	78	0	0.078	0.011	0.989	8.83	0
	100	17	0.100	0.014	0.986	8.86	109.0
	125	26	0.125	0.018	0.982	8.89	166.1
	150	29	0.150	0.021	0.979	8.92	184.7
	175	32	0.175	0.025	0.975	8.96	202.9
	200	34	0.200	0.028	0.972	8.98	215.1
3 rd cyc.	171	0	0.171	0.024	0.976	8.95	0
	180	10	0.180	0.025	0.975	8.96	63.4
~	~	~	~	~	~	~	~
4 th cyc	540	0	0.540	0.076	0.924	9.45	0
	550	15	0.550	0.078	0.922	9.47	89.9
	650	46	0.650	0.092	0.908	9.62	291.6
	750	55	0.750	0.105	0.895	9.76	320.1
	1,200	66	1.200	0.169	0.831	10.51	356.8
	1,300	67	1.300	0.183	0.817	10.68	356.4

Note: Insert units in column headings as necessary.

*The Deviator stress computation shown is based on taring the loading system so that the load ring reading is the deviator load value. $\sigma = (\text{Col. } 3 \times \text{LRC})/A'$.

Computed Data Note: original data edited; data plotted on Fig. 15-8

Lateral pressure on test specimen $\sigma_3 = 98 \text{ kPa}$ (1 kg/cm²)

Maximum deviator stress (from stress-strain curve) $\sigma = 357.5$

Maximum value of vertical stress $\sigma_1 = \sigma + \sigma_3 = 455.5$ (Used in Fig. 15-9)

Figure 15-7

Stress-strain data for a resilient modulus triaxial test for cell pressure shown.

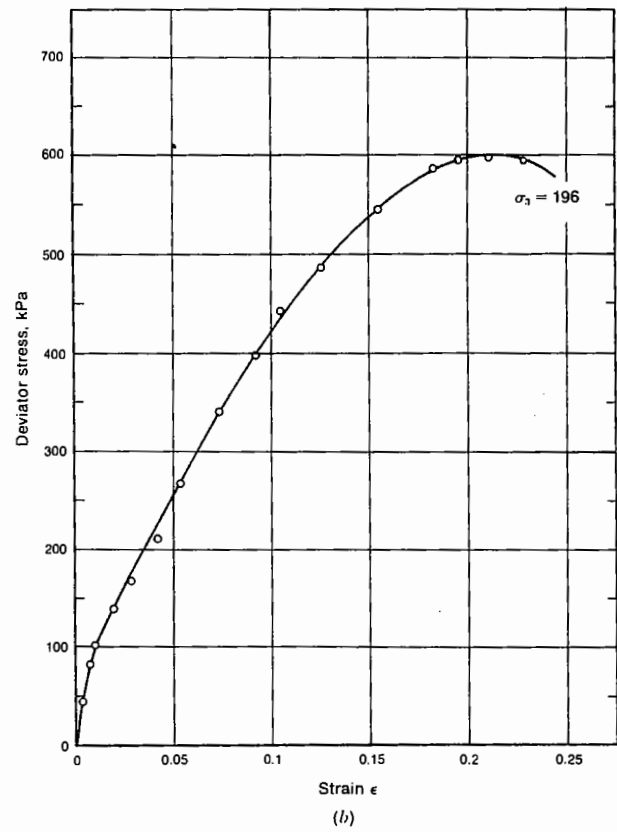
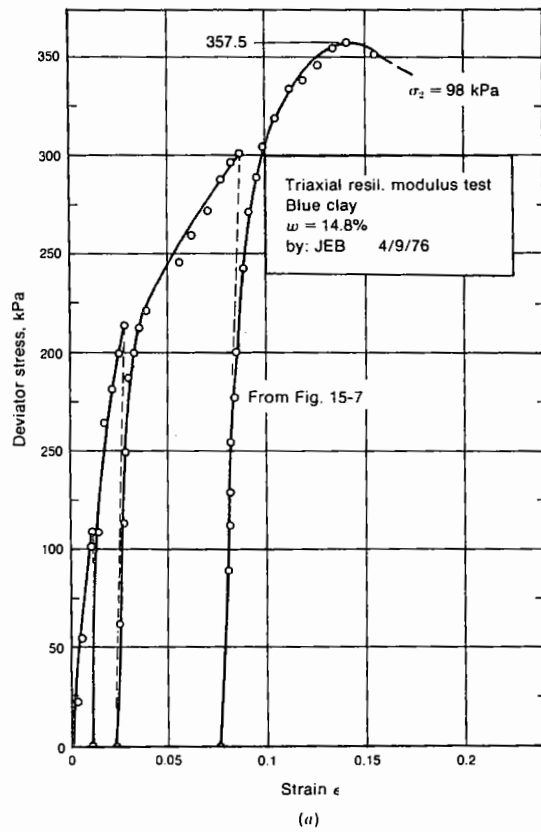


Figure 15-8
Stress-strain plots.
(a) Resilient modulus test from edited stress-strain data in Fig. 15-7.

(b) Second test (cell pressure of 196 kPa and data not shown) to obtain maximum value of deviator stress.

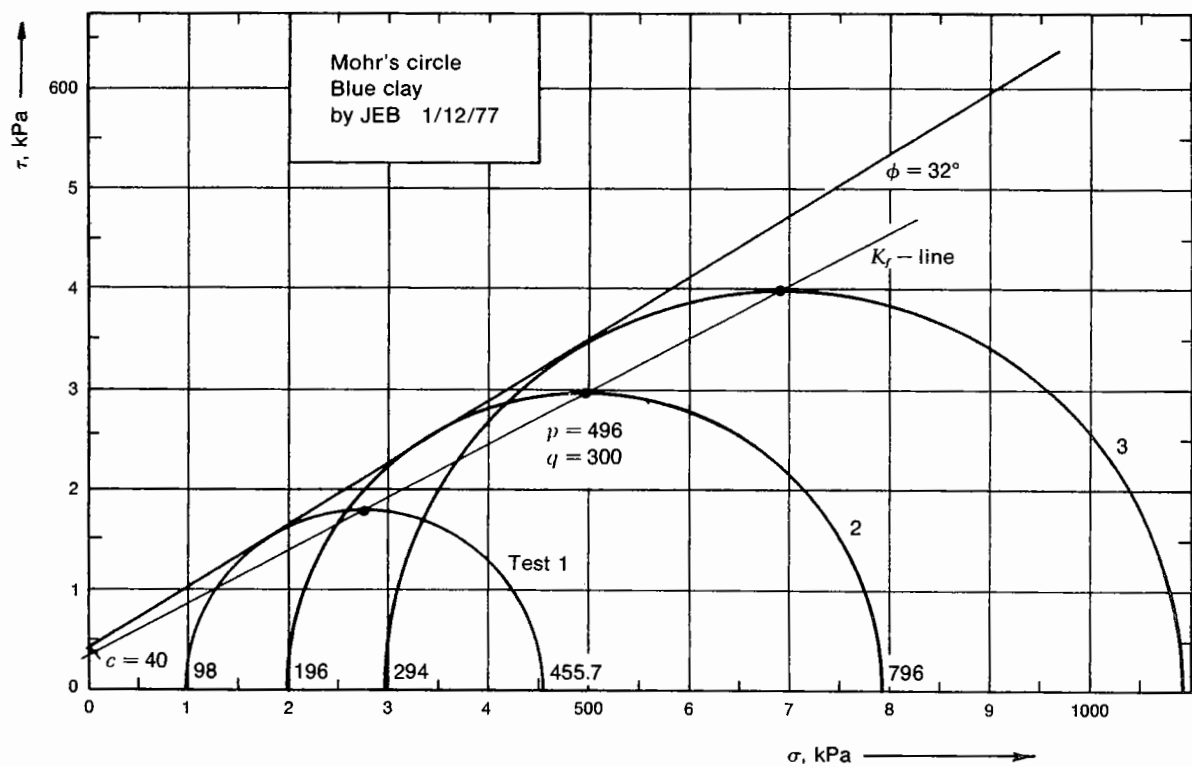


Figure 15-9
Mohr's circle for data of Fig. 15-8 and additional test not shown.

Experiment No. 16

TRIAXIAL TEST—WITH PORE-PRESSURE MEASUREMENTS

References

See Experiment No. 15

Bishop, A. W., and D. J. Henkel (1962), "The Measurement of Soil Properties in the Triaxial Test," 2d ed., Edward Arnold Ltd., London.

Chan, C. K., and J. M. Duncan (1967), A New Device for Measuring Volume Changes and Pressures in Triaxial Tests on Soils, *Mater. Res. Stand., ASTM*, July, pp. 312-313.

Skempton, A. W., (1954), The Pore Pressure Coefficients A and B, *Geotechnique*, London, vol. 4, no. 4, December, pp. 143-147.

Spence, R. A., and T. E. Glynn (1962), Shear Characteristics of a Marine Clay, *J. Soil Mech. Found. Div., ASCE, SM 4*, August, pp. 85-107 (describes equipment to obtain strength parameters of clay soil).

Objective

To present the basic procedures for obtaining pore-water pressure and/or volume changes during a triaxial shear test.

Equipment

Compression machine (preferably strain-controlled)

Triaxial cell

Pore-pressure apparatus (see Fig. 16-1) or pressure transducer and electronic readout equipment.

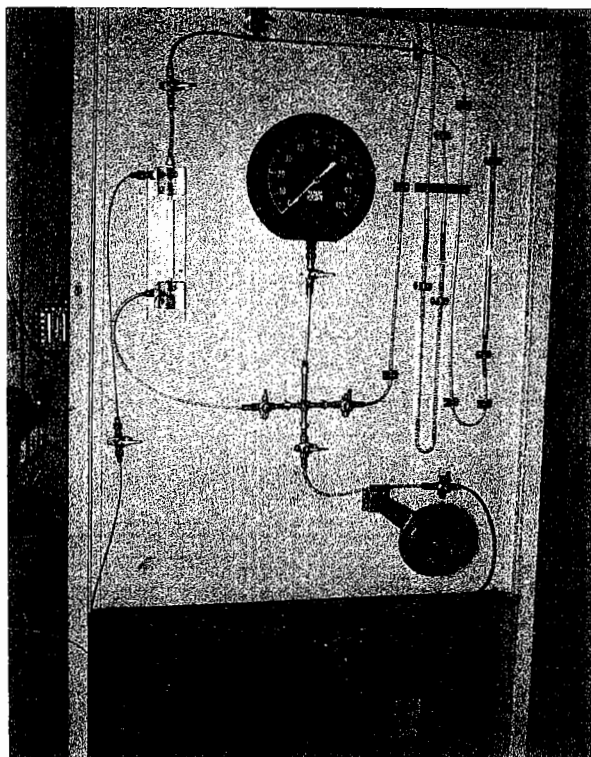
Specimen mold, rubber membranes, rubber binding strips, and porous stones

Calipers

Sample-trimming device (optional)

Figure 16-1

Pore-pressure panel (noncommercial). Valve on lower left connects to 3-mm-diam nylon tubing from soil sample inside cell. Plastic device on left is null indicator. The U tube on right can measure negative pore pressures. Burette on extreme right measures volume change when valve above null indicator is opened and valve from indicator into cross beneath pressure gage is closed. Large disk at bottom right is hydraulic metering device used to add water to system to force mercury column in null indicator back to zero position (visually). In use all the tubing must be saturated. All critical valves must be nondisplacement type. Use plastic tubing connections if mercury contamination is possible.



General Discussion

Read the "General Discussion" of Experiments No. 14 and No. 15.

It has long been recognized that the solution of Coulomb's equation

$$\tau = c + \sigma_n \tan \phi$$

does not really determine the soil parameters c and ϕ for partially to fully saturated soils when they are determined from the methods outlined in Experiments No. 15 and No. 17. The reason for this is that a soil mass is a compressible skeleton of solid particles. The voids are filled with water in a two-phase system (when $S = 100$ percent) or water and air for a three-phase system ($S \neq 100$ percent) or air only if the degree of saturation is zero percent. Shear stresses are carried by the soil skeleton since water and air have negligible shear strength (unless the water is in the form of ice).

Normal stresses, the σ_n of Coulomb's equation, are carried by both the solid particles and the pore fluid.

In all types of soil shear tests (and it may be noted that nearly all soil strength tests are shear tests), the shear strength is generally determined in terms of total stress, which includes intergranular stresses and pore-fluid stresses, as

$$\sigma_t = \sigma' + u \quad (16-1)$$

where σ_t = total pressure

σ' = intergranular or grain-to-grain stress (also termed effective stress)

u = pore-fluid pressure (which acts equally in all directions and can be considered on horizontal and vertical, or *normal*, stress planes)

In terms of effective stresses Coulomb's shear stress equation becomes

$$\tau = c' + \sigma' \tan \phi' \quad (16-2)$$

where c' and ϕ' are *effective* stress parameters obtained when using the effective stress σ' obtained from Eq. (16-1). It should be carefully noted that the pore pressure of interest may be either a static condition or, more often for soil below the water table, the temporary (transient) condition of *excess pore pressure*, sometimes termed Δu in the literature, which is caused by soil structure reorientation due to changes in soil stresses from increased (and sometimes decreased) foundation loads.

One may obtain the effective stresses σ' in two ways:

1. Perform the shear test so that the excess pore pressure $\Delta u \rightarrow 0$. The pore pressure due to structure reorientation can never be zero, but if the reorientation is at a slow enough rate the effect on the soil parameters is not significant.
2. Perform the shear test so that the excess pore pressure Δu can be measured.

The *consolidated-drained test* described in Experiment No. 15 is an attempt to satisfy the condition of

$$\Delta u \rightarrow 0$$

Although this test is very time-consuming (with a duration of days to possibly one or more weeks), it is probably the most accurate—particularly if the degree of saturation $S < 100$ percent.

For saturated soils, direct pore-pressure measurements during the test provide the most rapid effective stress parameters and with careful attention to detail are probably as accurate as any method.

Skempton (1954) proposed the following equation to describe the excess pore pressure for either saturated or nonsaturated soils:

$$\Delta u = B(\Delta \sigma_3 + A\{\Delta \sigma_1 - \Delta \sigma_3\}) \quad (16-3)$$

where Δu = change in pore pressure due to any incremental increase in confining pressure $\Delta \sigma$ and deviator stress $\Delta \sigma_1 - \Delta \sigma_3$. Note that $\Delta \sigma$ and σ are often used interchangeably, and the reader should look at the context of discussion to see what is meant.

A, B = pore-pressure coefficients (or parameters)

From an inspection of Eq. (16-4), one can determine the B coefficient by applying a stress to a sample in a triaxial cell of σ_3 (or $\Delta \sigma_3$) and measuring the resulting pore pressure. For saturated soils, B should be 0.98 to 1.00. It may be slightly less than 1.00 because of test inaccuracies and limitations. With the B coefficient established, the application of the deviator stress will enable an evaluation of the A coefficient.

The pore-pressure increase u due to application of σ_3 and the deviator stress $\sigma_1 - \sigma_3$ can be used in Eq. (16-3) to obtain σ' or in the field to estimate pore-pressure increases for, say, embankment construction for dams, levees, roads, etc. Actually, for soils of $S < 100$ percent, the evaluation of soil parameters ϕ and c will be questionable using Eq. (16-4) and Eq. (16-3) since it is obvious that the distribution of pore pressure across the failure plane in the soil specimen may not be uniform. This has also been validated on several construction projects where piezometers were installed to measure and compare the design pore pressure with the field pore pressure. This test limitation is similar to that of the permeability tests—without it, one has no idea of what the pore pressure can be; with it, one has something on which to base a design.

For soils of $S = 100$ percent, the use of Eq. (16-4) will yield much more reliable results. Actually, Eq. (16-4) is more useful in the laboratory in another way (for saturated tests); that is, one can detect whether a test sample is saturated by applying a lateral pressure in several increments and computing the B coefficient. If values of 0.98 to slightly over 1.00 are obtained, one can assume that the soil sample is saturated.

The measuring of pore pressures can be accomplished by two basic techniques:

1. By connecting a pressure transducer to the saturation line¹ and, in turn, connecting the output of the transducer to a digital (or other type) voltmeter. Pressure transducers can be obtained with very small volume displacements required to activate full-scale (capacity) output. This is an essential requirement if valid pore-pressure readings are to be taken.
2. By using some type of null-pressure indicator device (typical as shown in Fig. 16-1 and schematic of Fig. 16-2). These devices are commercially available, but if you make your own you will know (a) how it works, and (b) what its limitations are. A device similar to that shown in Fig. 16-2 can be made at a nominal cost

¹Some laboratories have used the method of inserting a large needle in the sample at what it is hoped will be the location of the shear failure. The needle is then connected to an outlet in the cell so that pore-pressure measuring equipment can be attached. There are three good reasons for not recommending this procedure:

- a. An automobile tire inner-tube patch must be applied so that the needle joint is leakproof; this requires considerable ingenuity.
- b. The failure surface is not often known.
- c. The sample is often disturbed by the insertion of the needle, which must be at least 3 mm ($\frac{1}{8}$ in) in diameter in order to be fitted to keep out small soil grains, which will plug it otherwise.

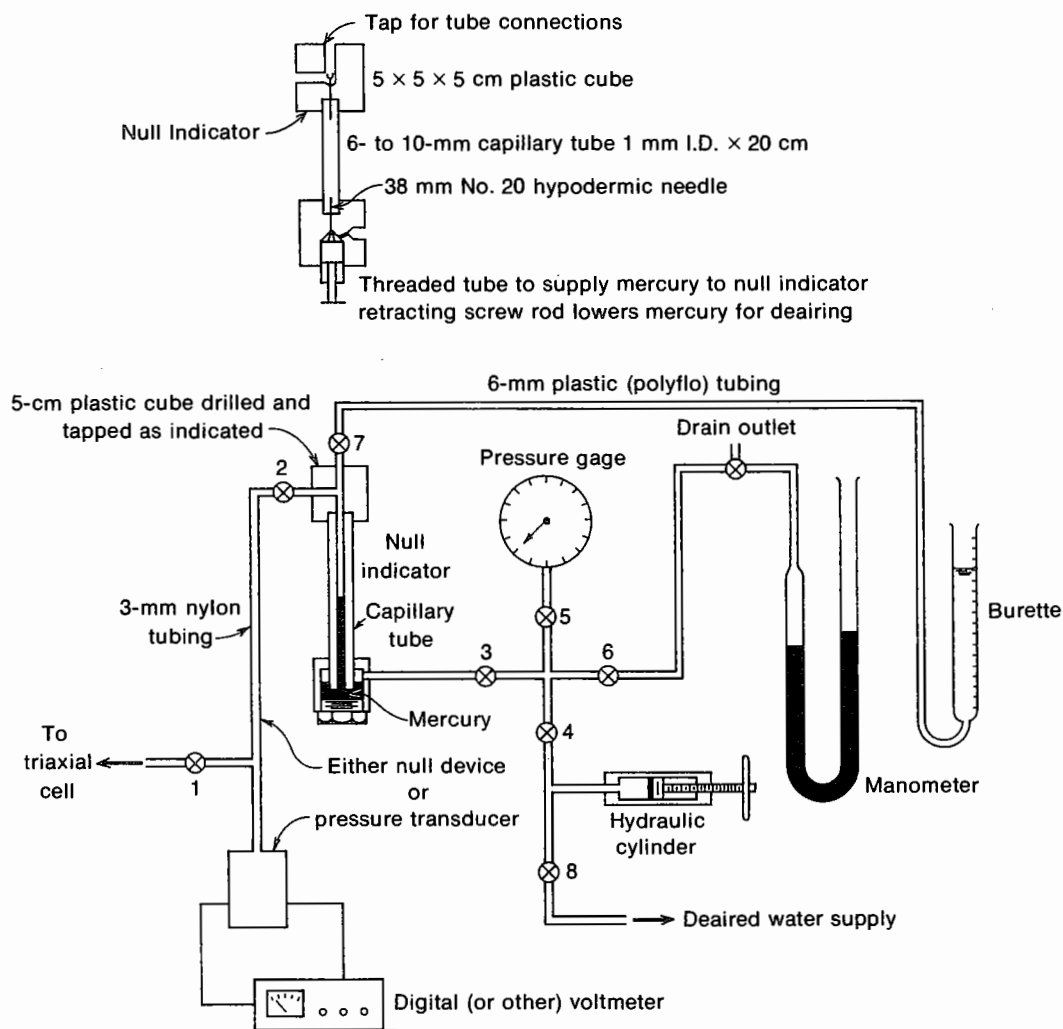


Figure 16-2

Line details of pore-pressure measuring apparatus. Drill connecting holes in null device so that slight tilting will allow air bubbles to float out. In use all tubing must be saturated (use deaired, distilled water).

(depending on who pays the machinist or who does the machining required). The following is a list of essential parts:

- 8 valves (open-shut which do not displace any volume when opening/closing)
- 1 pressure gage (20-cm diam \times 5 to 7 kg/cm² range to 0.05 kg/cm²/div)
- 1 hydraulic cylinder with screw control (fabricate in shop), 100 ml capacity
- 1 glass U tube for a manometer to measure negative pore pressures
- 1 burette (100-ml) to measure volume changes
- 1 null indicator (build in shop); refer to Fig. 16-2 for line details:
 - 2 pieces of 5 \times 5 \times 5 cm plastic refer to Fig. 16-1 for drilling and tapping.
 - 1 piece of capillary tube 1 mm ID \times 6 to 10 mm OD and 20 cm long
 - 2 No. 20 hypodermic needles 38 mm in length (stainless steel)
 - 1 threaded plastic rod (use convenient threads) about 10 cm long with threads on about 6 cm of rod.
 - 1 metric scale to observe and calibrate the mercury column
- $\frac{1}{2}$ kg mercury metal for null indicator and manometer (U tube)
- Nylon tubing 3 mm \times 3 m (use nylon to withstand high pressures without expansion)

Plastic (polyflo) tubing 6 mm \times 3 m for locations where expansion is not critical. Sufficient fittings with 3- and 6-mm ($\frac{1}{8}$ - and $\frac{1}{4}$ -in) standard pipe threads. Be sure to use nylon fittings where there is any possibility of mercury contamination or the device will have an abnormally short service life.

The null indicator system operates in the following manner (refer to Fig. 16-2):

1. Deair the system and stabilize the mercury in the null indicator.
2. Connect the 3-mm nylon tubing (precharged with water) to the sample outlet.
3. Close all valves. Apply the cell pressure σ_3 . Now open valves numbered 1, 2, 3, 4, and 5 on the line drawing.
4. If the sample is saturated, the mercury will immediately drop in the null indicator due to the cell pressure; add water to the system with the hydraulic cylinder to bring the mercury column in the null indicator back to its original location. The pressure required to do this can be read on the dial of the pressure gage and should be very nearly equal to σ_3 in cell.
5. Apply the deviator load and visually control the mercury column at the initial mark using the hydraulic cylinder. Record the pressure gage readings along with the deviator load and deformation readings on the data sheet provided.

Inspection of the tubing diagrams of Figs. 16-1 or 16-2 indicates that volume changes can be taken either to determine the end-of-consolidation for consolidated tests or (if pore pressures are not taken) to measure volume change of the test sample under load (with valves 1, 2, and 7 open).

If it is known in advance that the soil structure will collapse during application of the deviator load, one may connect the sample outlet to the mercury manometer (U tube with valves 1, 2, 3, and 6 open) to measure negative pore pressures. If the sample collapses otherwise, the operator will need to apply vacuum with the hydraulic cylinder (and not recorded on pressure gage) to keep mercury from being drawn out of the null indicator into the sample side of the system.

If it is desired to measure pore pressures and volume changes simultaneously, or to measure volume changes occurring in a nonsaturated sample, two test modifications are required:

1. It will be necessary to use a liquid to develop the cell pressure, σ_3 . By filling the cell with a liquid (such as glycerine-water mixture) and monitoring fluid into or out of the cell during the test, with allowance for piston advance into the cell, volume changes can be computed. Lubricate the loading piston with silicone (stopcock) grease to reduce cell leakage.
2. Use a compensating device of some type to maintain constant cell pressure when flow into, or out of, the cell occurs. Figure 16-3 illustrates the line details of a device described by Bishop and Henkel (1962). The principles of this device will be described in the following paragraphs.

With reference to the datum shown (Fig. 16-3) the three energy heads, h_1 , h_2 , and h_3 can be obtained. The energy heads h_1 and h_2 are the differential locations of two mercury heads connected by a length of flexible tubing such as 6 mm OD nylon tube. The top of the mercury in container *B*, referenced by head h_2 , is in contact with the chamber fluid and the cell pressure σ_3 through a T connection to the cell. The other side of the T connection is connected to the cell-fluid reservoir. With this configuration, the cell pressure is maintained by the difference between h_1 and h_2 , or in equation form,

$$\sigma_3 = (h_1 - h_2)\gamma_m \quad (a)$$

where γ_m = unit weight of mercury. Now if the sample inside the membrane

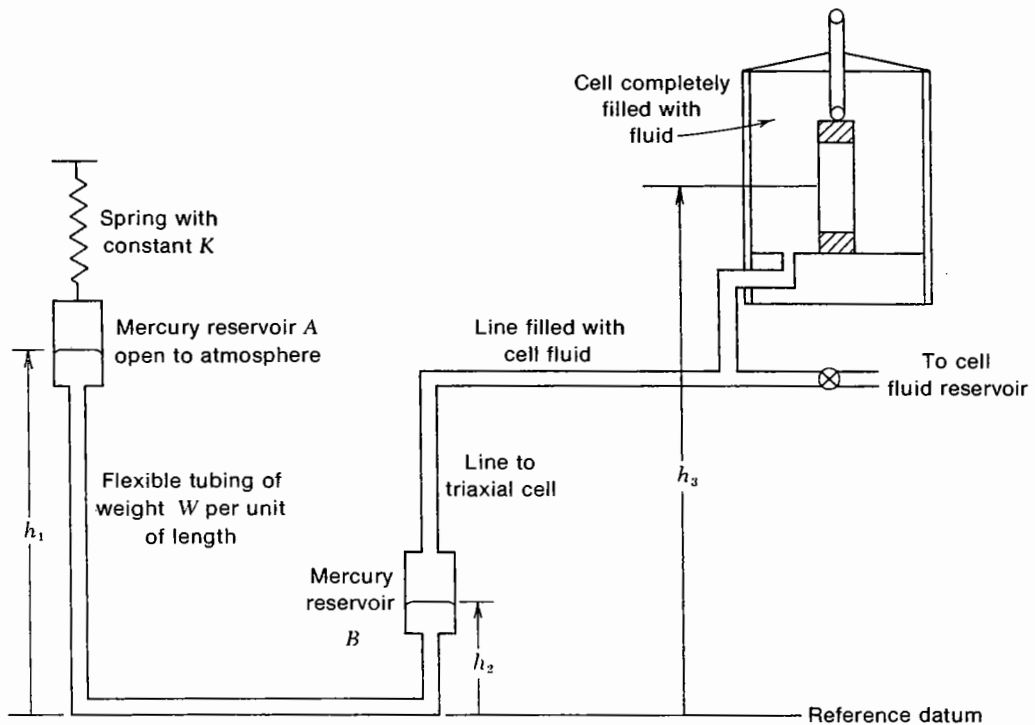


Figure 16-3
Schematic of compensating device to measure volume change for unsaturated samples during the triaxial test. Note that allowance must be made for the load piston advance into the cell.

changes in volume (say, reduces), the cell pressure decreases and the mercury at the h_2 location (in container B) rises an amount Δh . This will change the value of $h_1 - h_2$ by Δh and if nothing is done to compensate for this, the cell pressure will become

$$\sigma'_3 = (h_1 - h_2 - \Delta h)\gamma_m \quad (b)$$

As we wish to hold $\sigma_3 = \text{constant}$ (hopefully to $\pm 0.01 \text{ kg/cm}^2$ or less), something must be done to the system to compensate for the change of Δh above. What can be done is to put the container of mercury (A of Fig. 16-3) of h_1 on a spring. Thus, when h_2 rises, mercury runs out of container A, decreasing its weight, and the spring then shortens. If the spring constant K is of a proper value, the shortening of the spring will be enough to exactly balance the system pressure against the change of mercury reference height h_2 caused by Δh .

The spring constant can be evaluated as follows:

The change in weight of container A due to Δh in B is

$$\Delta W = \gamma_m A_B \Delta h \quad (c)$$

where A_B is the area of container B (for convenience, container A should have the same cross-section area). The net weight change must include a small length of the tubing connecting A and B, which must be lifted when the spring shortens an amount of $(\Delta L)W$; therefore, the net weight is

$$\Delta W_{\text{net}} = \gamma_m A_B \Delta h - (\Delta L)W \quad (d)$$

The spring constant multiplied by ΔL is also the net weight change, or

$$K \Delta L = A_B \gamma_m \Delta h - (\Delta L)W \quad (e)$$

or

$$K = \frac{A_B \gamma_m \Delta h}{\Delta L} - W \quad (f)$$

To obtain an expression for ΔL , consider the following:
If cylinders A and B are the same diameter (or size),

$$\Delta h_2 = -\Delta h_1 = \Delta h$$

and if both cylinders (A and B) are stationary, the change in pressure is

$$\sigma_3 + \Delta \sigma_3 = (h_1 - \Delta h - h_2 - \Delta h) \gamma_m + (h_2 + \Delta h - h_3) \gamma_w$$

But σ_3 can be obtained by writing a manometer-type-fluid static equation for the system:

$$\sigma_3 = (h_1 - h_2) \gamma_m + (h_2 - h_3) \gamma_w$$

from which

$$\Delta \sigma_3 = -(2\gamma_m - \gamma_w) \Delta h \quad (g)$$

Now, if we raise the cylinder A by the length ΔL ,

$$\sigma_3 + \Delta \sigma_3 = (h_1 + \Delta L - \Delta h - h_2 - \Delta h) \gamma_m + (h_2 + \Delta h - h_3) \gamma_w$$

or

$$\Delta \sigma_3 = \Delta L \gamma_m - (2\gamma_m - \gamma_w) \Delta h \quad (h)$$

and if $\Delta \sigma_3 = 0$,

$$\Delta L \gamma_m - (2\gamma_m - \gamma_w) \Delta h = 0$$

and the desired expression for ΔL becomes

$$\Delta L = \left(2 - \frac{\gamma_w}{\gamma_m} \right) \Delta h \quad (i)$$

Substituting this value into E (f), we obtain the desired expression for the spring constant:

$$K = \frac{A_B \gamma_m}{2 - \frac{\gamma_w}{\gamma_m}} - W \quad (16-4)$$

where A_B = area of cylinders A and B

γ_m = specific weight of mercury = 13.53 g/cm³

γ_w = specific weight of chamber fluid

W = weight per unit of length of flexible tube connecting A and B . (This weight is about 0.001 kg/mm for 6-mm-OD plastic tubing and, hence, can be neglected for all practical purposes.)

K = desired spring constant (force/length)

Example

Let $A = B = 5.08$ cm; therefore, area = 20.3 cm^2

Chamber fluid = water-glycerine mixture of $\gamma_w = 0.9 \text{ g/cm}^3$

Substituting into Eq. (16-4) obtain

$$K = \frac{(20.3)(13.53)}{2 - 0.9/13.53} = 142 \text{ g/cm}$$

Figure 16-4 can be used to evaluate qualitatively the merits of drained and undrained testing of cohesive and cohesionless soil samples.

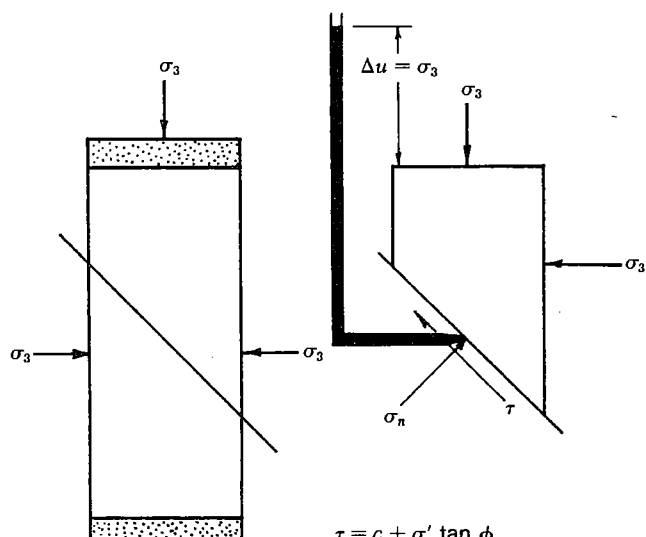


Figure 16-4

Pore pressure effects in an undrained test. It is evident that an undrained test on a saturated cohesionless soil can only measure the shear strength of the triaxial membrane.

$$\begin{aligned} \tau &= c + \sigma' \tan \phi \\ \text{If } \Delta u &= \sigma_3; \sigma' = 0 \\ \Delta u &= \sigma_3 + \Delta \sigma; \sigma' = 0 \\ \therefore \tau &= c \quad \text{for cohesive soils} \\ \tau &= 0 \quad \text{for cohesionless soils} \\ \text{If } \Delta u &\neq \sigma_3, \\ \text{then } \tau &= c + \sigma' \tan \phi \end{aligned}$$

Procedure

A. COHESIONLESS SOIL¹

1. Attach a rubber membrane of the proper diameter (at least 5 cm for sand²) to the base platen (refer to Fig. 15-2), which has already been fastened to the base of the cell, using rubber strips or bands. To provide a more impervious joint, the base platen may be lightly coated with silicone grease to increase the seal between the membrane and platen.

Some persons measure the wall thickness of the rubber membrane prior to attaching it, so that an area adjustment can be made for the initial area of the sample, but for ordinary computations this introduces (in the author's opinion) a fictitious precision into the results.

Place a porous stone on the base of the platen. For the tests in this section, the stone must be saturated. Boiling the top and bottom stones in distilled water will increase the degree of saturation and is recommended. Merely storing the stones underwater, even for a long period of time, may not deair them sufficiently to achieve the desired degree of saturation.

¹For illustrative student laboratories a very silty sand should be used so that measurable pore pressures can be developed and consolidation, etc., is rapid.

²The largest grains should be less than one-sixth the diameter of the test sample.

2. Weigh a container of dry sand so that the sample density can be obtained and approximately duplicated for succeeding tests. Note that to obtain a given density it may be necessary to mix the soil with a known amount of water to produce a specific water content. Damp cohesionless soil can be compacted to a limited extent—dry material is difficult to nearly impossible.
3. Place a specimen mold around the rubber membrane and fold the top portion of the membrane down over the mold. Do not fold, however, if the mold is so large that the membrane will be torn in the process. Membrane stretchers may be used for forming the sample if the correct size is available. If a split-barrel type, which can be attached to a vacuum, is used, it may be necessary to use some silicone grease along the split to effect a vacuum seal. For both the split stretchers and other types of stretchers using vacuum, it may also be necessary to use threads or small strips of filter paper between the membrane and the stretcher walls so that the vacuum will be effective the full height of the membrane stretcher.
4.
 - a. Deair the water in the saturation reservoir by connecting it to a vacuum for 5 to 10 min.
 - b. Open the saturation valve and allow water to flow into the bottom of the membrane for a depth of about 2 cm; then close the valve.
 - c. Place sand through the water in the bottom of the membrane, using a tamper to obtain the desired density to a depth just under the water surface.
 - d. Open the saturation valve and allow about 2 cm more of water; then add sand as before. Repeat until the sample is just below the top of the mold. Use a pipette and remove most of the excess water.
 - e. Now place the saturated top porous stone on the sample.
5. Place the top platen on the porous stone. It may be necessary to coat the perimeter of the top platen with silicone grease to increase the seal. Roll the membrane off the mold and onto the top platen and seal to the platen with rubber strips or bands. Take a small level and check the level of the top platen.
6. Attach the tube from the top platen to the vacuum outlet in the base of the cell and apply a vacuum of 150 to 200 mm of mercury. Simultaneously, open the saturation line (so you do not desaturate the sample—observe to see if more water appears to be going out of the sample than is going in and, if so, reduce the vacuum). Close the saturation line.
7. Now remove the specimen mold and observe the membrane for holes and obvious leaks. If any are found, the sample must be remade using a new membrane.
8. Obtain four height measurements approximately 90° apart and use the average value for the initial specimen height L_o . Take two diameter readings 90° apart at the top, at midheight, and at the base using a pair of calipers. Take these measurements to the nearest 1 mm. Compute the average diameter of the specimen at each height location and then compute a final average specimen diameter as

$$d_{av} = \frac{d_t + 2d_m + d_b}{4}$$

where d_t is the average diameter based on the two top measurements, etc. Compute the corresponding value of initial sample area A_o using the average diameter just computed.

9. Place the lucite cylinder on the cell base, being sure the base is free of soil grains so that an airtight seal can be obtained. Place the cell in the compression machine and just barely make load contact of the loading piston and the loading bar (or crosshead of the compression machine).
10. Apply a predetermined lateral pressure σ_3 (preferably in multiples of 10 kPa

or 0.5 kg/cm² for computational ease) using compressed air or other fluid (such as water and glycerine mixture) and simultaneously reduce the vacuum on the interior of the sample to zero. In passing, for long-duration tests, the rubber membrane will eventually leak regardless of type of cell fluid used. Leakage can be reduced using a glycerine-water mixture and two membranes with silicone grease between the membranes.

11. If it is desired to measure volume change to detect end-of-consolidation, connect the saturation line to the volume-change burette. A plot of volume change vs. time should indicate when consolidation is nearly complete. Approximate end-of-consolidation may be obtained by monitoring the deflection dial and when the sample ceases shortening, consolidation should be about complete.
12. After consolidation is complete, connect the pore-pressure null indicator¹ to the saturation-line outlet and observe the stationary position of the null indicator.
13. When the chamber pressure was applied, it could be seen that the load indicator indicates an upward force which is due to the difference in weight of the piston rod and the cell pressure acting upward on the piston base. Carefully adjust the triaxial cell until piston-sample contact is reestablished and then zero the load indicator (dial gage). This step makes for direct reading of the deviator load and more convenient computations.

At this point open the other exit line to the soil sample if a "drained" test is to be performed.

14. Attach a deformation dial (reading to 0.01 mm) to the machine so that the deformation of the sample can be obtained. Set the dial gage to zero; then manually compress and release the dial plunger several times, observe the zero reading, and readjust to zero if necessary. Recheck the load dial for zero reading. Check the chamber-pressure gage. Check the null indicator.
15. Set the compression machine to the desired strain rate (between 0.5 and 1.25 mm/min, as specified by the instructor).
16. Turn on the compression machine and take simultaneous load- and deformation-dial readings and pore-pressure readings from the pore-pressure gage. Remember to keep the pore-pressure gage activated by observing the null indicator and adding water from the hydraulic cylinder to maintain the mercury column at the initial position.

Generally, for samples of 63 to 100 mm in diameter, deformation readings can be taken at each 0.5 mm of deformation (50 dial divisions of the 0.01 mm dial range). Take readings until the load holds constant and then falls off or to slightly beyond the estimated 20 percent strain value. If this yields a very large number of load-deformation readings, do the computations (see Procedure C) on enough of the readings to define adequately the stress-strain curve and its peak value. Be sure to observe the chamber-pressure gage throughout the test and do not let the chamber pressure vary by more than 0.05 kg/cm².

17. After the sample fails, shut off and/or reverse the compression machine, take off the sample load, and gradually release the chamber pressure so that the null indicator does not lose mercury.
18. Remove the lucite cover and the tested sample. Prepare a new specimen to the same approximate density (within 0.2 to 0.4 kN/m³) and make two additional tests. This completes the laboratory work for a cohesionless soil; refer to Procedure C to complete the report.

¹If it is desired to determine whether saturation is 100 percent, prior to connecting the sample to the volume-change burette, connect it directly to the null indicator using a σ_3 less than the test value. Now incrementally increase σ_3 and record the resulting pore pressure. Compute the B coefficient, and if it is very nearly 1.00, the sample can be assumed to be saturated.

B. COHESIVE SOIL

1. Prepare two or three tube samples of adequate L/d ratio of 2.2 to 2.5. It may be necessary to use a sample trimmer to form the samples if they are of smaller diameter than the tube samples.

Alternatively, compact three or four samples at some constant water content and compaction effort using the Harvard miniature compaction apparatus. This procedure will yield samples of approximately 36-mm diameter. These samples may not be saturated, however, and if it is desired to test saturated samples, it may be necessary to use tube samples.

2. For the samples which have been prepared, obtain four height measurements approximately 90° apart and use the average value for the initial specimen height L_o . Take two diameter readings 90° apart at the top, at midheight, and at the base using a pair of calipers. Take these measurements to the nearest 1 mm. Compute the average diameter of the sample at each height location and then compute a final average specimen diameter as

$$d_{av} = \frac{d_t + 2d_m + d_b}{4}$$

where d_t is the average diameter based on the two top measurements, etc. Compute the corresponding value of initial sample area A_o using d_{av} from the above equation.

3. Take the correct-size membrane stretcher and membrane and fit the membrane smoothly into the stretcher, folding the ends of the membrane over the ends of the stretcher. Attach a vacuum tube to the membrane stretcher and apply a vacuum. If there are no leaks, the membrane will form a smooth cover on the inside of the stretcher under the vacuum. It may be necessary to place some small threads or filter-paper strips between the membrane and the walls of the stretcher to make the vacuum effective when stretching the membrane tight prior to the next step.
4. Depending on the sample, either lightly lubricate the membrane with petroleum jelly, powder it with teflon powder (inert substance), or as a last resort, moisten it with water for ease of insertion of the sample into the membrane.
5. Cut thin strips of filter paper (at least four) long enough to fit under the porous stone of the lower platen and extend above the membrane stretcher when it has been slipped onto the lower platen. Fold the tops of the filter-paper strips back over the rubber membrane and fasten out of the way with a rubber band. Be sure the porous stone is saturated.

Insert the sample into the membrane and seat it on the porous stone. Release the vacuum on the membrane and roll the bottom part down onto the platen and seal it with rubber bands or strips. Silicone grease on the perimeter of the platen may increase the seal. Attach the upper saturated porous stone, slip the filter-paper strips between it and the upper platen, and then seat the upper platen. Use the same procedure for the lower platen.

6. Connect the top-platen line to the vacuum line but do not apply a vacuum. Flood all the lines into and out of the sample; then close off the top-platen line. Connect the saturation line to the pore-pressure null indicator.
7. Place the cover on the lucite cell and place the cell in the compression machine. Bring the load bar into contact with the load piston until a load just flickers on the load dial.
8. Apply part of the incremental load of σ_3 and observe the pore pressure. Do this at least twice and compute the B coefficient to see if the sample is saturated. If volume change is to be observed (also recommended if consolidated tests are to be made), disconnect from the null indicator and connect to the volume-change burette.

9. Apply the full value of σ_3 and observe the volume-change burette. Make a plot of volume-change vs. elapsed time. When the semilog plot resembles one from the consolidation test (Experiment No. 13), the consolidation of the sample can be assumed to be complete. Now reconnect to the null indicator. Note that the strips of filter paper are to speed up the consolidation process.

From this point on, refer to steps 13 to 18 of Procedure A and to all of Procedure C as the work is identical.

C. COMPUTATIONS

1. Compute the unit strain from the deformation-dial readings as

$$\epsilon = \frac{\Delta L}{L_o}$$

and fill in the appropriate column of the data sheet. Also compute the adjusted area

$$A' = \frac{A_o}{1 - \epsilon}$$

and place this in the appropriate column of the data sheet.

2. Compute the deviator load using the load-dial readings. If a load ring is used, the deviator load is

$$P = DR \times \text{load-ring constant}$$

where DR is the load-dial reading in units of deflection. Put these data in the appropriate column of the data sheet.

3. Compute the deviator stress

$$\sigma = \frac{P}{A'}$$

and fill in the appropriate column of the data sheet.

4. Plot a curve of unit deviator stress vs. unit strain (on the abscissa) and obtain the stress at the peak point unless the stress at 20 percent strain occurs first. Show this value of deviator stress on the graph. Plot all the tests on the same graph, with each curve identified for the corresponding lateral pressure σ_3 .
5. With the maximum deviator stress from step 4, compute the major principal stress for each test as

$$\sigma_1 = \sigma_3 + \sigma$$

Also compute the pore pressure corresponding to the maximum deviator stress from a plot of deviator load vs. pore pressure. Compute the effective pressure σ'_3 and the major effective principal stress, σ'_1 as:

$$\sigma'_3 = \sigma_3 - u \quad \sigma'_1 = \sigma_1 - u$$

6. Plot Mohr's circles for both *total* and *effective* principal stresses on the same set of axes for all the tests. Fit a tangent (stress envelope) to each set of circles and measure the slope to obtain the apparent and "true" angles of internal friction and the two values of soil cohesion. If the testing has been done on cohesionless soils, the cohesion should be quite small. If it is not, discuss

reasons for the divergence. Be sure to show all values on the graph neatly identified.

7. Compute the apparent and "true" angle of internal friction ϕ for the cohesionless-soil tests using Eq. (15-2) and compare with the results obtained in step 6 above.
8. Show a neat line drawing of the laboratory test setup as part of the report regardless of type of soil tested.
9. If cohesive soils have been tested, be sure to comment in the report and show on the Mohr's circle plot the type of test performed (that is, CD test, CU test, U test, etc.) and whether the soil is saturated or unsaturated.
10. Compute the initial tangent modulus and the secant modulus using the origin and a curve intercept at 50 percent of ultimate strength and compare values. Estimate Poisson's ratio using the method of step 12 of Experiment No. 15.

Experiment No. 17

DIRECT-SHEAR TEST

References

- ASTM D3080-72
ASCE (1960), Research Conference on Shear Strength of Cohesive Soils, Boulder, Colorado.
ASTM (1964), Symposium on Laboratory Shear Testing of Soils, *ASTM STP no. 361*.
ASTM (1952), Direct Shear Testing of Soils, *ASTM STP no. 131*.
Lee, K. L. (1970), Comparison of Plane Strain and Triaxial Tests on Sand, *J. Soil Mech. Found. Div., ASCE, SM 3*, May, pp. 901–923.

Objective

To familiarize the student with a procedure for rapidly determining the strength parameters (ϕ and c) of a soil

Equipment

Direct-shear device (see Fig. 17-1)
Calipers
Small level

General Discussion

The direct-shear test imposes on a soil the idealized conditions shown in Fig. 17-2. That is, the failure plane is forced to occur at a predetermined location. On this plane there are two forces (or stresses) acting—a normal stress due to an applied vertical load P_v and a shearing stress due to the applied horizontal load P_h . These stresses are simply computed as

$$\sigma_n = \frac{P_v}{A} \quad (17-1)$$

$$\tau = \frac{P_h}{A} \quad (17-2)$$

where A is the nominal area of the specimen (or of the shear box) and is usually not corrected for the change in sample area caused by the lateral displacement of the sample under the shear load P_h . These stresses should satisfy Coulomb's equation of Experiment No. 15,

$$\tau = c + \sigma_n \tan \phi \quad (15-1)$$

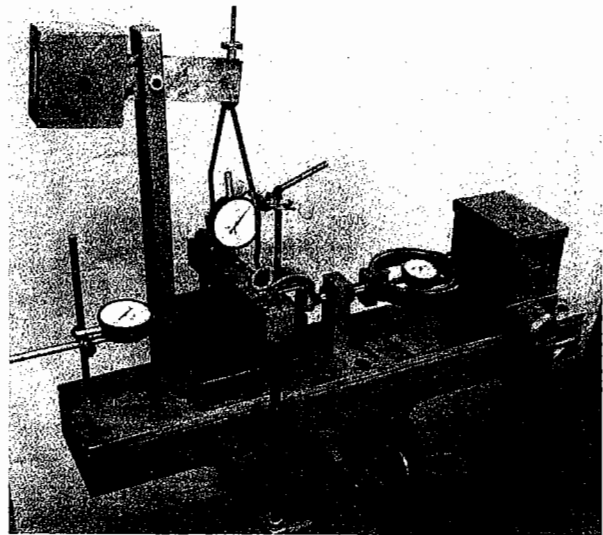
As there are two unknown quantities (c and ϕ) in Eq. (15-1), two values, as a minimum, of normal stress σ_n and shear stress τ will be required to obtain a solution.

Since the shear stress τ and normal stress σ_n have the same significance as when used in a Mohr's circle construction, rather than solving a series of simultaneous equations for c and $\tan \phi$, one may plot on a set of coordinate axes the values of τ versus σ_n from several tests (generally with τ on the ordinate), draw a line

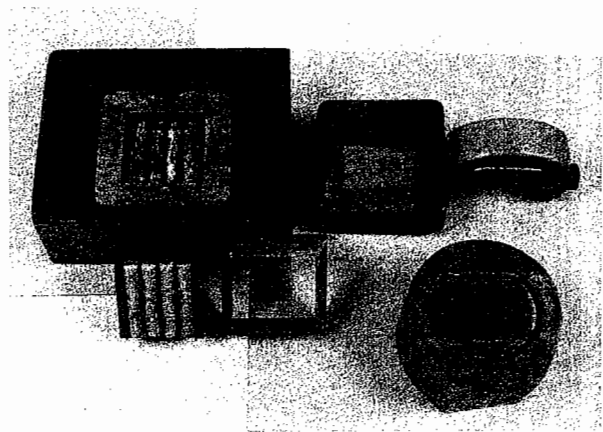
Figure 17-1

Direct shear equipment.

(a) One type of direct shear machine which may be hand-driven or motorized (as shown). All machines are similar in details: Load ring and load-ring dial on right. Center dial is resting on vertical load yoke cross-bar and measures sample consolidation/expansion during shear. Dial gage on left measures shear deformation.



(b) Two shear boxes. Either round or square type bolts to saturation box which is removable from machine [see under loading yoke in (a)]. Note use of serrated porous stones. Square sample trimmer also shown. Sample thickness is limited to about 20 mm; weight of round load block and upper half of shear box shown as 1778 g. Note that this can be tared using the appropriate counterweight adjustment—see large block attached to vertical load yoke in (a).



through the resulting locus of points, or the average locus of points, and establish the slope of the line as ϕ and the τ -axis intercept as the cohesion c . This yields a graphical solution of Eq. (15-1).

For cohesionless materials, the cohesion should be zero by definition and Eq. (15-1) becomes

$$\tau = \sigma_n \tan \phi \quad (17-3)$$

Test inaccuracies and surface-tension effects of damp cohesionless materials often give a small value of cohesion (apparent), which should be neglected unless it is more than 10 to 15 kPa. If the cohesion value is large and the soil is a cohesionless material, the reason for the large value should be investigated.

The direct-shear test was formerly quite popular. Then, as the state of the art advanced, it tended to become less popular for several reasons:

1. The area of the sample changes as the test progresses, but may not be very significant as most samples "fail" at low deformations.
2. The actual failure surface is not plane, as is assumed or as was intended from the way the shear box was constructed, nor is the shearing stress uniformly distributed over the "failure" surface, as is also assumed.
3. The test uses a small sample, with the result that preparation errors become relatively important.

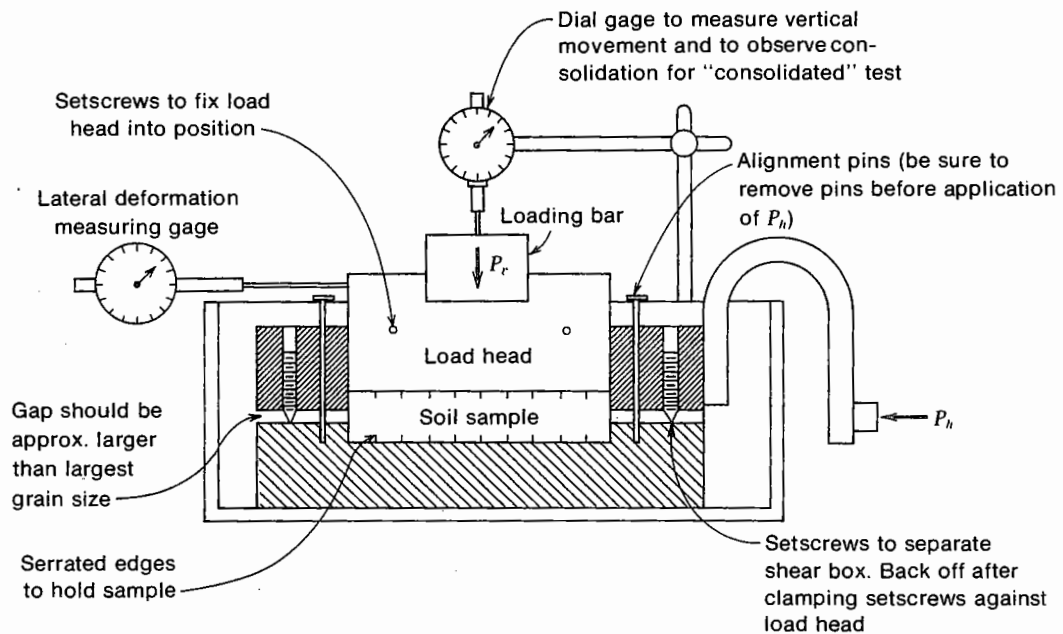


Figure 17-2
Line details of shear box/test

4. The size of the sample precludes much investigation into pore-water conditions during the test.
5. Values of modulus of elasticity and Poisson's ratio cannot be determined.
6. Triaxial test apparatus was developed.

With a further advance in the state of the art, however, the direct-shear test appears to be regaining much of its former popularity. Some of the reasons for this are as follows:

1. The triaxial test is, relatively, much more difficult to perform and interpret—especially if pore pressures are taken.
2. The size of the sample makes it not so time-consuming to perform consolidated-undrained and consolidated-drained tests since drainage time is relatively short, even for materials of low coefficient of permeability, when the drainage path is quite short.
3. Square sample boxes were introduced so that the reduction in area during the test can be easily accounted for, if desired. The use of square boxes is relatively recent, and many older machines currently in service still use round sample boxes.
4. The direct-shear machine is much more adaptable to electronic readout equipment, so that an operator is not required on continuous duty for consolidated-drained tests, which may be of several days' duration.
5. It has been found that the soil parameters ϕ and c obtained by the direct-shear-test method are about as reliable as triaxial values¹ (probably this is more the result of an operator problem than of the equipment being equal in performance). This does not mean to imply that the triaxial test is undesirable; rather, if soil parameters are all that are desired, the direct-shear values have usually been found to be quite acceptable. And, as has been pointed out, there is some information, such as pore-water behavior during shear, which cannot be evaluated using the direct-shear test.

¹On sands above 35° the ϕ -values from a direct shear test may be 1 to 4° larger than in a triaxial test (see Lee, 1970). Below 35° the ϕ -values are about the same.

Direct-shear tests may be categorized as follows:

1. *Unconsolidated-undrained or U-tests.* Shear is begun before the sample consolidates under the normal load P_v . If the soil is cohesive and saturated, excess pore pressures may develop. This test is analogous to the unconsolidated-drained triaxial test.
2. *Consolidated-undrained test.* The normal force is applied, and the vertical dial movement is observed until settlement stops before the shearing force is applied. This test is something between the consolidated-drained and the consolidated-undrained triaxial tests.
3. *Consolidated-drained test.* The normal force is applied, and the shear force is delayed until all settlement stops; the shear force is then applied so slowly that no pore pressures develop in the sample. This test is analogous to the consolidated-drained triaxial test.

For cohesionless soils, all three of the above tests will give the same results, whether the sample is saturated or unsaturated and, of course, if the shearing rate is not extremely rapid. For cohesive materials, the soil parameters are markedly influenced by the test method and the degree of saturation, and whether the soil is normally or over consolidated. Generally, two sets of shear-strength parameters are obtained for overconsolidated soils—one set for tests using normal loads less than the preconsolidation pressure and a second set for normal loads greater than the preconsolidation pressure. Where a preconsolidated soil is suspected, it may be necessary to perform six or more tests to ensure that the appropriate shear-strength parameters are obtained.

Procedure *This Will Be a Class Project*

A. COHESIONLESS SOIL—LABORATORY WORK

1. Weigh a large dish of dry (or wet with water content accurately known) sand of sufficient quantity to do at least 3 tests at the same density.
2. Carefully assemble the shear box (back off any sample box separation and clamping screws) and fix into position. Obtain sufficient dimensions that the sample density can be computed. Obtain the sample cross-section area A .
3. Carefully place the sand in the shear box to about 5 mm from the top and place the loading block (and included porous stone) on top of the soil. Take the small level and level the loading block.

Weigh the container of sand to determine weight of the material making up the sample. Obtain a reference thickness of the soil sample by marking the loading block at several points around the perimeter with respect to the shear box.

4. Apply the desired normal load P_v and attach the vertical displacement dial gage (reading to 0.01 mm/div). Remember to include the weight of the loading block and upper half of shear box as a part of P_v .¹

For consolidated tests, observe the vertical displacement dial and commence the test only after the settlement has halted. For cohesionless materials this will be almost immediately after application of P_v .

5. Separate the two parts of the shear box by advancing the spacing screws in the upper half of the shear box. The space should be slightly larger (by eye) than the largest soil grains in the sample. Now set the load block by tightening the setscrews provided for that purpose in the sides of the upper half of the shear

¹Some shear machines (including author's) allow taring this along with load hanger or yoke so the added weight is the effective normal load.

box. Next back off the spacing screws so they clear the bottom half of the shear box; at this time the normal load, top half of shear box, and load block are all carried by the soil sample.

6. Attach the dial gage (0.01 mm/div) to measure the shear displacement.
7. For a saturated test, saturate the sample by filling the shear box with water and allow a reasonable time for saturation to take place.

Be sure the porous stones in the shear box are saturated if the soil to be tested contains any moisture.

8. Start the horizontal (shear) loading and take readings of load dial, shear displacement, and vertical (volume-change) displacements. If a strain-controlled test is performed, take these readings at horizontal displacements of

5, 10, and every 10 or 20 horizontal dial displacement units

Use a strain rate on the order of 0.5 to not more than 2 mm/min. Do not use too fast a strain rate or the shear load may peak between readings. The strain rate should be such that the sample "fails" in about 3 to 5 min.

9. Remove the sand from the shear box and repeat steps 1 through 8 on at least two additional samples and to a density within 5 g and not more than 10 g of soil quantity used in first test. Be sure that the sand goes into the same volume by using the reference marks of step 3.

In step 4 use a different value of P_v for each test (suggest doubling the exterior weights, say, 4, 8, and 16 kg + weight of load block for three tests or 5, 10, 20 kg, etc).

B. COHESIVE SOIL—LABORATORY WORK

1. Carefully trim three or four samples of the same size (and, hopefully, of the same density) from the larger sample block, tube sample, or other sample source. Use the sample cutter so that the size can be accurately controlled. Any sample with a weight appreciably different from the others should be discarded and another sample trimmed. [What constitutes "appreciable" compared to the size of the sample (order of 5 cm square \times 20 to 25 mm thick) will be a matter of judgement.]

Note: You may need six samples if the soil is undisturbed and preconsolidated. Keep the samples in a controlled humidity while trimming, preparing the shear machine, and taking care of other test details.

2. Back off the separation and clamping setscrews in the top half of the shear box and assemble the two parts. Be sure that the porous stones are saturated unless you are testing a dry soil.

Measure the shear-box dimensions to compute the area.

3. Carefully place the soil sample in the shear box. It should just fit into the box and fill it to about 5 mm from the top.

Place the loading block in place, the normal load P_v , and attach the vertical dial gage.

For a consolidated test, monitor the vertical dial gage as for a consolidation test (Experiment No. 13) to determine when consolidation is complete.

4. Carefully separate the shear-box halves using a gap slightly larger than the largest soil grains present, back off the separation screws and clamp the loading head in place using the setscrews for that purpose.

Be sure the normal load reflects the normal load + the weight of the load block and top half of shear box.

Be very careful in separating the shear box when testing soft clay that material is not squeezed out between the two box halves—use of small vertical loads and/or consolidation prior to box separation may be required.

5. Attach the shear-deformation dial gage, set both vertical and horizontal dial gages to zero. Fill the shear box with water for saturated tests and wait a reasonable time for saturation to be complete.
6. Start the horizontal (shear) loading and take readings of load dial, shear displacement, and vertical (volume-change) displacements. If a strain-controlled test is performed, take these readings at horizontal displacements of

5, 10, and every 10 or 20 horizontal dial displacement units

Use a strain rate on the order of 0.5 to not more than 2 mm/min. Do not use too fast a strain rate or the shear load may peak between readings. The strain rate should be such that the sample "fails" in 5 to 10 min unless a CD test is being run.

The strain rate for CD tests should be such that the time for failure to occur t_f is

$$t_f \cong 50t_{50} \quad (17-4)$$

where t_{50} is the time for 50 percent consolidation to occur under the normal load P_v . If t_{50} is not readily obtainable

$$t_f = 35t_{60} = 25t_{70} = 12t_{90}$$

may be used.

A plot of vertical dial reading vs. log time as for a consolidation test can be made to determine when the soil has completed consolidation. When P_v is very large, it may be necessary to apply the load in increments rather than all at once for reasons outlined in Experiment No. 13.

7. Remove the soil and take a water-content sample. Repeat steps 2 through 6 for two or more additional tests. If soil is preconsolidated and you use six tests, be sure to use a range of normal loads for three on each side of the preconsolidation pressure.

C. COMPUTATIONS

The following computations are applicable to either cohesionless or cohesive soil.

1. Compute the nominal normal stress as

$$\sigma_n = \frac{P_v}{A}$$

where A = cross-section area of shear-box soil sample

P_v = total normal load including load block and top half of shear box.

2. Plot the horizontal displacement δ_h vs. horizontal shear force P_h to obtain the best value of ultimate shear force¹ to compute the shear stress as:

$$\tau = \frac{P_{h(\text{ultimate})}}{A}$$

Note: One may use the residual shear force (value somewhat less than the ultimate at a displacement beyond that for ultimate shear force) in this computation for obtaining the residual-strength parameters.

¹Alternatively, plot horizontal-displacement dial units vs. load dial units as in Fig. 17-4 to obtain the maximum shear force.

DIRECT SHEAR TEST (Cohesive Soil, Cohesionless Soil)

Data Sheet 21

Project Direct Shear test [edited data] Job No. ~Location of Project Bradley University Boring No. ~ Sample No. ~Description of Soil Med. Coarse Sand Depth of Sample ~Tested by JEB & RGL Date of Testing 1/12/77Soil state (~~wet~~, dry) Soil sample (disturbed, ~~undisturbed~~)

Data to Obtain Sample Density if not an Undisturbed Sample

Initial wt. container + soil = 1376.2 g
 Final wt. container + soil = 1236.0
 Wt. of soil used = 140.2

Water Content Data

Wt. wet soil + cup = ~
 Wt. dry soil + cup = ~
 Wt. of cup = ~
 Wt. of water = ~
 Wt. of dry soil = ~
 Water content, $w\%$ = ~

Shear specimen data

Sample Dimensions:

Diam. or side = 5.08 x 5.08 cmHt. = 3.42 cmArea = 25.81 cm²Vol. = 88.26Density: γ_{wet} = ~ γ_{dry} = 15.62 kN/m³Normal load = 5 kgNormal stress σ_v = 19.0 kPa
(1.376 N/div)Loading rate = 0.50 mm/min Load ring constant = 0.1406 kg/div.

	Vert. dial reading ($\times 0.01$ mm)	Vert. displace. ΔV , ()	Horiz. dial reading ($\times 0.01$ mm)	Horiz. displace. ΔH ()	Corr. area A'	Load ring dial reading	Horiz. Shear force, ($\frac{N}{\text{div}}$)	Shear stress τ , kPa
	0	0	0	0	—	0	0	0
	+0.5		10			14		
	+2.5		20			19		
	+3.0		40			24		
	+2.0		60			25		
	+2.0		75			26		
	+1.5		100			29		
	+1.5		150			30		
	+1.5		175			31		
	+1.5		200			31		
	+1.5		250			29.5		
	+1.5		300			26		

Note: See Fig. 17-4 for Plot. +V = expansion of sample

Note: Insert units in column headings as necessary.

"For square samples, may use corrected specimen area at failure as $A' = A_0 - b \Delta H$ to compute σ_v and τ .

Figure 17-3 Direct shear-test data (cohesionless soil).

- Plot the value of shear stress τ vs. σ_n for the tests. Fair a straight line through the plotted points (Fig. 17-5). Be sure to use the same scale for both the ordinate (τ) and the abscissa. Obtain the cohesion (if any) as the intercept with the ordinate axis and measure the slope of the line to obtain ϕ .

If you plot the residual shear stress vs. σ_n the residual shear-strength parameters are obtained.

- On the graph of δ_h vs. P_h and using the same horizontal displacement scale make a plot of vertical displacement vs. δ_h (as Fig. 17-4). This plot will display volume change versus shear displacement. Make appropriate comments in your report concerning the magnitude and shape of the plot.
- In your report make appropriate comments on the shear-strength parameters obtained.

Consider whether you should have used a corrected area in computing the shear stress (and normal stress) or whether the results are conservative or unconservative without the correction for area.

Comment on why it is necessary in Experiments No. 14 through No. 16 to plot strain vs. stress to obtain the maximum stress where the maximum shear stress can be obtained from a plot such as Fig. 17-4 in this soil test.

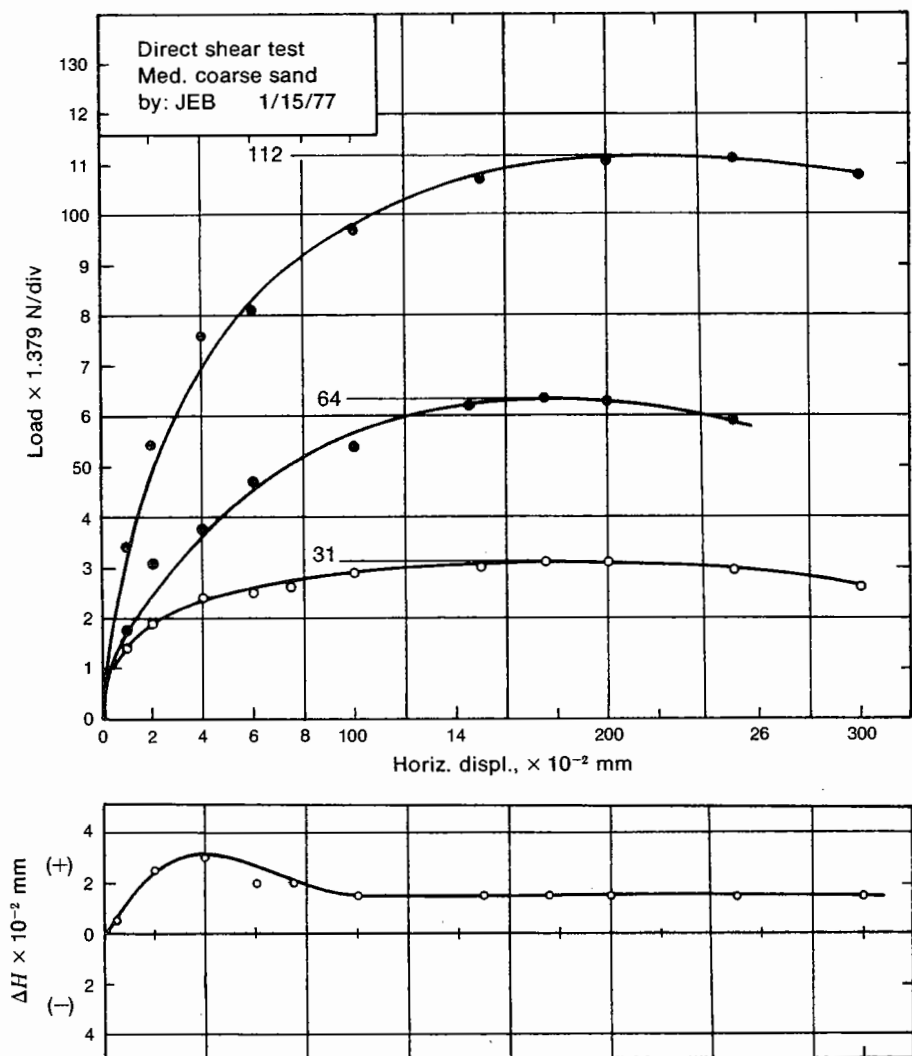


Figure 17-4

Plot of shear and volume change vs. horizontal displacement (note type of units for time economy).

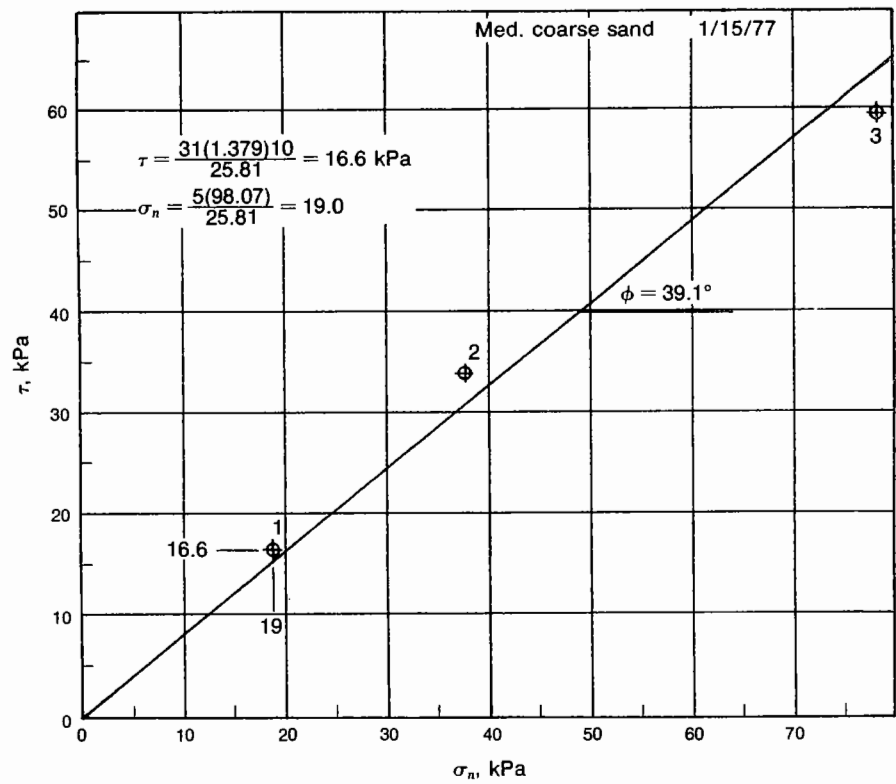


Figure 17-5
Plot of shear stress vs. normal stress to obtain soil parameter(s).

Experiment No. 18

RELATIVE-DENSITY DETERMINATION

References

ASTM D2049-69

ASTM (1973), Relative Density Involving Cohesionless Soils, *ASTM STP no. 523*.

Objective

To determine the state of density of a cohesionless soil with respect to its maximum and minimum densities. The student will be introduced to the limitations of this type of test. The student will be also introduced to a simple method of obtaining a compaction density for a cohesionless soil.

Equipment

Standard compaction mold or a calibrated volume measure as used in concrete-mix design laboratories

Hand or mechanical vibration equipment

General Discussion

The impact method for obtaining the moisture-density curve of Experiment No. 9 does not work well for cohesionless soils (either wet or dry). Some persons obtain a control density in the laboratory for this type of material by filling a standard compaction mold in several layers, confining each layer in some manner (Fig. 18-1), and vibrating the mold by rapping it sharply on the sides with a rubber mallet. The largest density value obtained from several trials is then taken as the control criterion for the job.

A somewhat better criterion might be obtained by expressing the field control in terms of the *relative density* D_r of the soil. This has been defined by Terzaghi as a fraction equation of void ratios of the soil in its loosest state (e_{\max}), in the natural soil state (e), and in the densest possible state (e_{\min}):

$$D_r = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \quad (18-1)$$

It is often more convenient, however, to use the unit weight of the soil since the void-ratio computation requires the use of the specific gravity of the soil, which may not be known. Using the unit weights of the soil, the relative density can be expressed as

$$D_r = \frac{\gamma_f - \gamma_1}{\gamma_2 - \gamma_1} \frac{\gamma_2}{\gamma_f} \quad (18-2)$$

where γ_f = density of the soil in its natural state (field value)

γ_2 = densest value of unit weight obtained in the laboratory

γ_1 = loosest value of unit weight obtained in the laboratory

The problems in this test occur from properly defining the soil in its loosest and most dense states. As a consequence, some laboratories use a mechanical vibration device, and under an arbitrarily selected confining pressure and length of time of vibration, the soil is defined as being in the most dense state. Alternatively, vibrating a saturated soil has been used to obtain an arbitrary maximum

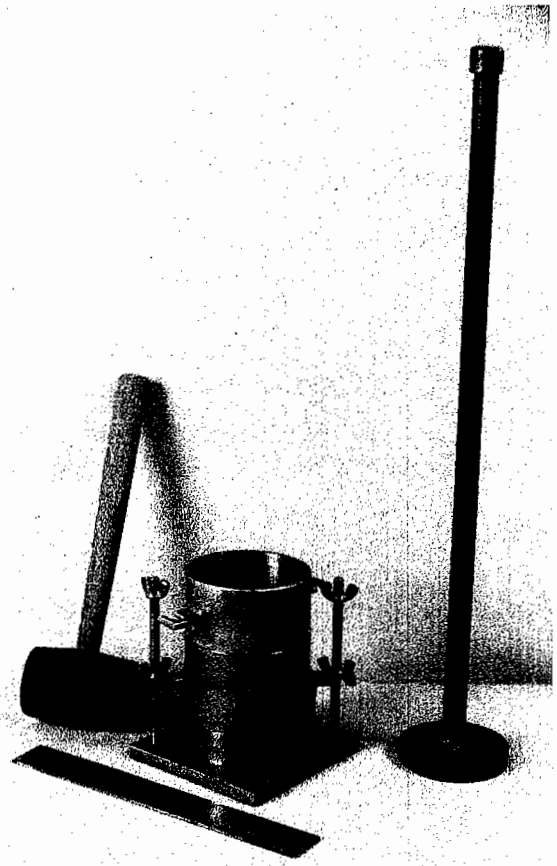


Figure 18-1

Relative density (or density) equipment suggested by author. Equipment consists in 1000-cm³ (SI mold) compaction mold, rubber mallet for vibration, steel straightedge for striking mold flush, and 10.2-cm diam plate welded to section of pipe to confine soil while striking mold sides with mallet.

density. In the author's method in the "Procedure" following, no increase in density was obtained using a saturated sand.

The loosest state is similarly defined, except it is desired to use no vibration while carefully pouring the soil into a volume measure. The loosest state has also been arbitrarily obtained by allowing the material to fall through water to form a specified volume.

The term *relative density* has no particular significance for cohesive soil. For these materials, *consistency* (i.e., soft, medium stiff, stiff, hard, etc.) and unit weight are generally used to describe the soil state.

At this time (1977) it appears, based on comments in STP 523 and current ASTM Committee D-18 discussions, that the relative density standard may be dropped due to the poor reproducibility of the test.

Procedure *This Will Be an Individual Project*

1. Each individual should take a representative sample of the oven-dry cohesionless soil from the stock supply furnished. Be sure to break any remaining lumps prior to use.
2. Obtain a standard compaction mold. Use the same mold for both the maximum- and minimum-density determinations so that you do not have to obtain mold dimensions and compute the volume.
3. Make three trials of maximum density by placing the material in the standard mold in five layers, confining the layer with a round steel block of at least 12 kg or obtain the assistance of another student to confine the soil using the confining plate (as shown in Fig. 18-1) while you rap the sides of the mold sharply 15 to

25 times (as specified by instructor) with a rubber mallet. Be sure to record the number of blows and layers in your report. After each trial carefully remix the test soil with that remaining in the container for the next test.

Use the largest density obtained (in your report discuss why), not the average, for the maximum density of the soil. Put this value on the blackboard or location specified by instructor.

4. Next, using the same soil and mold carefully pour the material into the mold, distributing the soil in a circular motion over the mold. Slightly overfill, then with a straightedge strike the excess with as little vibration as possible. Obtain the test weight and repeat at least two more times.

Use the smallest density obtained for the minimum density of the soil. Put this value on the blackboard beside your maximum density. Be sure to identify your work so the instructor will know who has not completed the assignment.¹

Note: There are no data sheets for this project.

Computations

1. Each student is to compute the standard deviation $\bar{\sigma}$ of the maximum and minimum class densities separately. Use \bar{X} = sample mean as the largest and least density values of the class and not the mean of the two populations.

$$\bar{\sigma} = \sqrt{\frac{\sum(\bar{X}^2 - X^2)}{N - 1}}$$

where N = number of density tests

X = any density-test value

2. Based on $\bar{\sigma}$, γ_{\max} , γ_{\min} , and a single arbitrary unit weight as the average of γ_{\max} and γ_{\min} , compute the largest and smallest expected error in D , for a single relative-density test on this soil.
3. Derive Eq. (18-2) from Eq. (18-1).
4. Comment on ways to improve test.
5. Comment on use of $N - 1$ instead of N in computing the standard deviation.

¹The instructor should put all density values on a stencil and make a copy for each student when all the density determinations have been made by the class.

Experiment No. 19

CALIFORNIA BEARING-RATIO (CBR) TEST

References

AASHTO T193-63
ASTM D1883-73

Objective

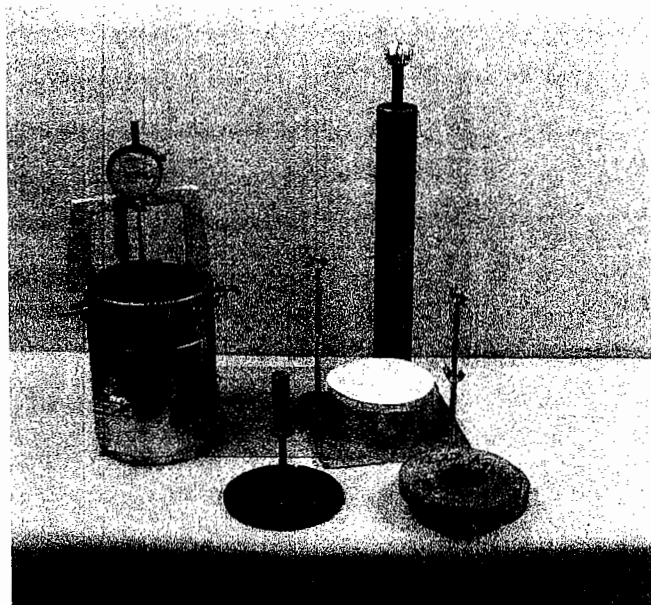
To introduce the student to a method of evaluating the relative quality of sub-grade, subbase, and base soils for pavements.

Equipment

CBR equipment (see Fig. 19-1) consisting of
15.2-cm diam \times 17.8-cm height compaction mold (or equivalent) with collar
Spacer disk 15.1-cm diam \times 6.14-cm height (or 5.1-cm height as available)
Compaction hammer (either 24.5 N or 44.5 N as designated by instructor)
Expansion measuring apparatus with dial gage reading to 0.01 mm
Surcharge weights
Compression machine equipped with CBR penetration piston (4.953-cm diam with cross-section area of 19.4 cm²) and capable of a penetration rate of 1.27 mm/min

Figure 19-1

CBR equipment. Mold, collar, and dial-gage holder for soaking test on left; 24.5-N compaction hammer, base plate, 5.1-cm-thick spacer disk in background. Swell plate with extension rod and surcharge weights for both soaking and penetration testing in foreground.



General Discussion

The California bearing-ratio test was developed by the California Division of Highways in 1929 as a means of classifying the suitability of a soil for use as a subgrade or base course material in highway construction. During World War II, the U.S. Corps of Engineers adopted the test for use in airfield construction.

The CBR test (ASTM terms the test simply a bearing-ratio test) measures the shearing resistance of a soil under controlled moisture and density conditions. The test yields a bearing-ratio number, but from the previous statement, it is evident this number is not a constant for a given soil but applies only for the tested state of the soil. In passing the test can be performed in the field on an in-place soil, but this is beyond the scope of this laboratory manual.

The CBR number (or, simply, CBR) is obtained as the ratio of the unit load (in pounds per square inch) required to effect a certain depth of penetration of the penetration piston (with an area of 19.4 cm²) into a compacted specimen of soil at some water content and density to the *standard unit load* required to obtain the same depth of penetration on a standard sample of crushed stone. In equation form, this is

$$\text{CBR} = \frac{\text{test unit load}}{\text{standard unit load}} 100 \quad \text{percent} \quad (19-1)$$

From this equation it can be seen that the CBR number is a percentage of the standard unit load. In practice, the percentage symbol is dropped and the ratio is simply noted as a number, such as 3, 45, 98. Values of standard unit load to use in Eq. (19-1) are as follows:¹

PENETRATION		STANDARD UNIT LOAD	
mm	in	MPa	psi
2.5	0.10	6.9	1,000
5.0	0.20	10.3	1,500
7.5	0.30	13.0	1,900
10.0	0.40	16.0	2,300
12.7	0.50	18.0	2,600

The CBR number is usually based on the load ratio for a penetration of 2.5 mm. If, however, the CBR value at a penetration of 5.0 mm is larger, the test should be redone (ordinarily). If a second test yields also a larger CBR number at 5.0 mm penetration, the CBR for 5.0 mm should be used.

CBR tests are usually made on test specimens at the optimum moisture value for the soil as determined using the standard (or modified) compaction test of Experiment No. 9. Next, using Method 2 or 4 of ASTM D698-70 or of D1557-70 (for the 15.2-cm diam mold) the specimens are made up using the compaction energy shown²:

Method	Blows	Layers	Hammer weight, N
D698: 2 (fine-grained soil)	56	3	24.5
4 (coarse soil)	56	3	24.5
D1557: 2 (fine-grained soil)	56	5	44.5
4 (coarse soil)	56	5	44.5

Two molds of soil are often compacted—one for immediate penetration testing and one for testing after soaking for a period of 96 h. The second specimen is

¹Values from ASTM Standards, Part 19, Test D1883-73.

²For standard mold and spacer disk; adjust blows/layer if using 5.1-cm thick spacer disk (see data sheet of Fig. 19-6).

soaked for a period of 96 h with a surcharge approximately equal to the pavement weight used in the field *but in no case is the surcharge weight less than 4.5 kg*. Swell readings are taken during this period at arbitrary selected times and at the end of the soaking period the CBR penetration test is made to obtain a CBR value for the soil in a saturated condition.

In both penetration tests for the CBR values, a surcharge of the same magnitude as for the swell test is placed on the soil sample. The test on the soaked sample accomplishes two things:

1. It gives information concerning expected soil expansion beneath the pavement when the soil becomes saturated.
2. It gives an indication of strength loss from field saturation.

Penetration testing is accomplished in a compression machine using a strain rate of 1.27 mm/min. Readings of load vs. penetration are taken at each 0.5 mm of penetration to include the value of 5.0 mm, and then at each 2.5-mm increment thereafter until the total penetration is 12.7 mm.

The CBR number is used to rate the performance of soils primarily for use as bases and subgrades beneath pavements of roads and airfields. The following table gives typical ratings.¹

CBR No.	General rating	Uses	CLASSIFICATION SYSTEM	
			Unified	AASHTO
0-3	Very poor	Subgrade	OH, CH, MH, OL	A5, A6, A7
3-7	Poor to fair	Subgrade	OH, CH, MH, OL	A4, A5, A6, A7
7-20	Fair	Subbase	OL, CL, ML, SC, SM, SP	A2, A4, A6, A7
20-50	Good	Base, subbase	GM, GC, SW, SM, SP, GP	A1b, A2-5, A3, A2-6
> 50	Excellent	Base	GW, GM	A1a, A2-4, A3

Many paving-design procedures are published in which one enters a chart with the CBR number and reads directly the thickness of subgrade, base course, and/or flexible pavement thickness based on the expected wheel loads.² Sometimes the CBR is converted to a subgrade modulus (also using charts) before entering the paving-design charts.

A. LABORATORY WORK

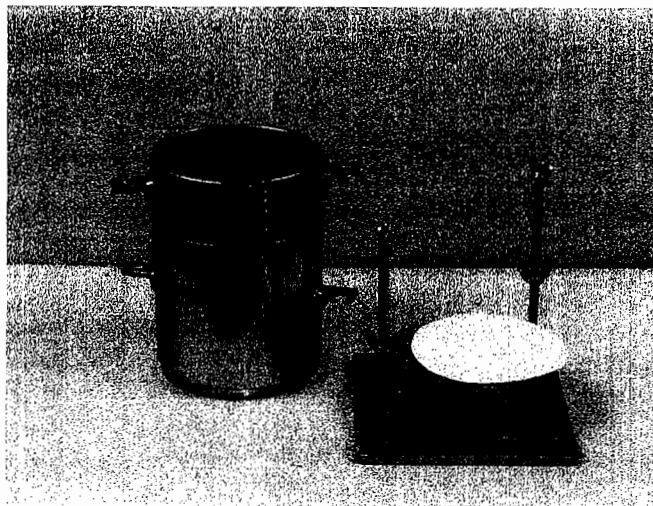
1. Prepare approximately 4.5 kg of fine-grained (—) No. 4 sieve material or 5.5 kg of 19 mm maximum-size material to the optimum moisture content of the soil as determined by the appropriate compaction effort (compaction test). If it is desired to cure the soil for a more uniform moisture distribution, mix the soil with the necessary percentage of moisture and store in a sealed container for about 12 to 24 h prior to the test.
2. Just before making the compacted mold of soil, take a representative water-content sample (at least 100 g for fine-grained soil).
3. Weigh the mold less base plate and collar.
4. Clamp the mold to the base plate, insert the spacer disk (the 15.1 × 5.1 cm solid round disk) in the mold and cover with a piece of filter paper (refer to Fig. 19-2).

¹"The Asphalt Handbook," The Asphalt Institute, 1962, chap. 5.

²E. J. Yoder, "Principles of Pavement Design," John Wiley & Sons, Inc., New York, 1959, chaps. 14 and 15. See also Development of CBR Flexible Pavement Design Method for Airfields—A Symposium, *Trans. ASCE*, vol. 115, 1950, pp. 453-589.

Figure 19-2

Mold, collar, base plate for test. Place spacer disk (5.1 cm thick shown) on perforated base plate and cap with a piece of filter paper so soil does not stick during compaction.



5. Compact the soil according to ASTM D698 or D1557 Method B or D for the soil used *or as specified by the instructor*. Take a final representative water-content sample from the remaining soil.
6. Remove the collar and trim the specimen smooth and flush with the mold. Using fine-grained soil, patch any holes formed when scraping the specimen flush with the mold. For coarse-grained soils, use any soil particles for patching which are smaller than the hole.
7. Remove the base plate and spacer disk, weigh the mold + compacted soil, and determine the wet unit weight.
8. Place a piece of filter paper on the base plate, invert the specimen (so the 5.1-cm gap is on top), and attach the base plate so the soil is in contact with the filter paper on the base.

For unsoaked specimens, do steps 9 to 11 next.

9. Place sufficient slotted weights (but not less than 4.5 kg) on the sample to stimulate the required overburden pressure.
10. Place the specimen in the compression machine and seat the piston using a seating load no greater than 4.5 kg (Fig. 19-3). Set load and penetration (or deformation) dials to zero.
11. Take penetration readings as outlined in the "General Discussion" and take the corresponding load-dial readings. Extrude the sample and take two additional representative water-content samples.

For soaked samples:

12. Place the perforated plate with adjustable stem on the compacted soil and apply sufficient additional slotted weights to obtain the desired surcharge within 2.2 kg but with a total surcharge weight of not less than 4.5 kg. Record the total surcharge weight (be sure to include the perforated plate as part of the surcharge weight). Also be sure to use a piece of filter paper between the perforated stem base and the soil to avoid the soil sticking to the stem base.
13. Immerse the mold and weights in a container of water so the water has access to both the top and bottom of the sample and attach the dial gage (reading to 0.01 mm) in its holding bracket (Figs. 19-4 and 19-5). Mark the mold where the bracket is placed so that it can be removed and replaced properly between readings.

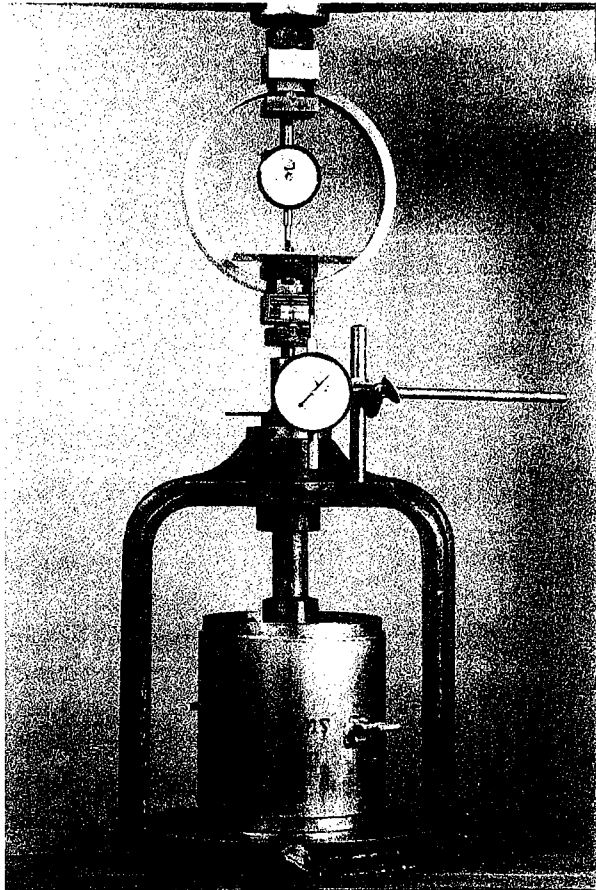


Figure 19-3
Setup for CBR penetration test. Note enlargement on penetration piston produces 19.35-cm² area and just clears hole or slot in surcharge weights of Fig. 19-1.

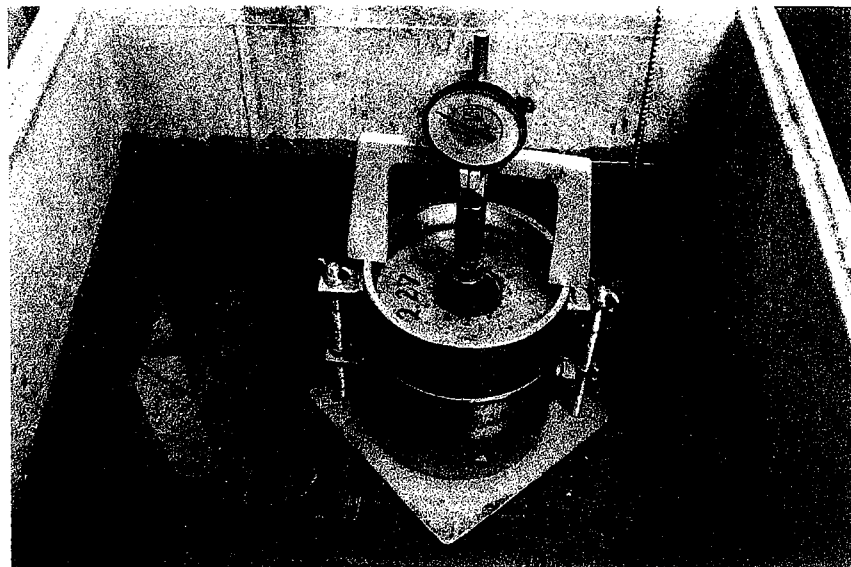


Figure 19-4
Setup for soaking test. Here water in laboratory sink covers surcharge plates. Note extension rod of swell plate extends through hole in surcharge plate and contacts extension rod of dial gage. Normally the contact points of the dial-gage holder are marked on mold collar so that gage can be removed and/or used for multiple tests.

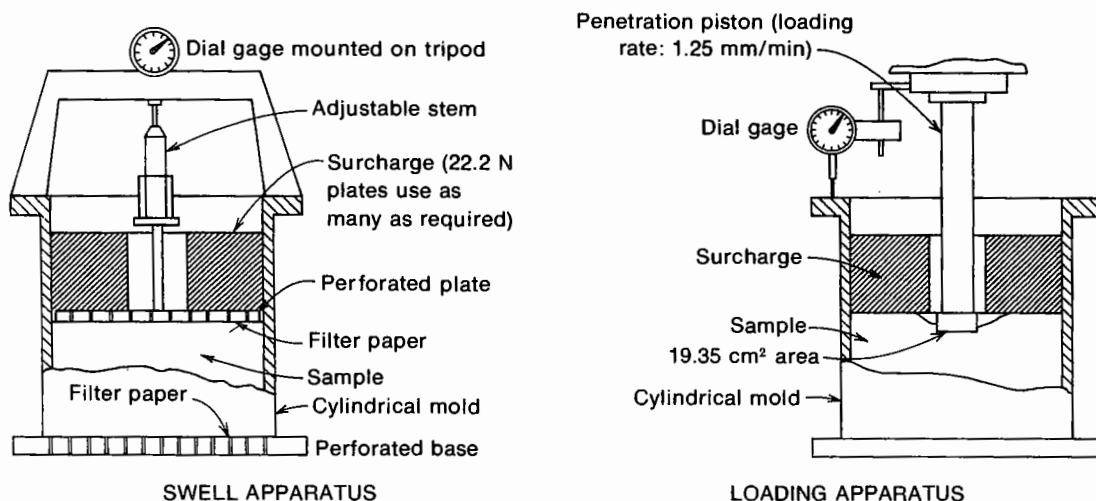


Figure 19-5

Line details of the CBR test. *Note:* Use 2.3 kg (= 22.2 N) for surcharge plate weights since you will be weighing them on your laboratory balances.

14. Set the swell gage to zero and record the time of the start of the test. Take readings at

0, 1, 2, 4, 8, 12, 24, 36, 48, 72, and 96 h

of elapsed time. The swell test may be terminated after 48 h if the swell-dial reading has been constant for the last 24 h.

15. At the end of 96 h of soaking, remove the sample and let it drain for 15 min. Blot the free surface water from the top surface of the sample with paper towels.
16. Weigh the soaked sample including the mold.
17. Do steps 9 to 11 (i.e., take penetration and load readings).
18. Take water-content samples of the soaked specimen as follows:
 - Two within the top 3 cm of soil
 - Two within the bottom 3 cm of soil
 - Two at the center of the sample

B. CALCULATIONS AND DATA PRESENTATION

1. Plot a curve of penetration resistance (the ordinate) in kilopascals (kPa) vs. penetration in millimeters for both the freshly compacted and the soaked samples. If the curve is not essentially linear through the origin, extend a line from the straight-line portion to intersect the abscissa. The difference between this value and zero penetration is a correction to apply to compute the CBR value. Curves for both samples (soaked and unsoaked) should be plotted on the same graph and clearly identified, together with the curve correction values, so that one may readily observe the effect of soaking the sample. Refer to Figs. 19-6 to 19-8 for computations and presentation of data. Use a sheet of graph paper from data sheet section.
2. Obtain the penetration resistance for 2.5 and 5.0 mm from the curve (using corrections from step 1 above, if necessary) and compute the CBR number.

Note: For student laboratory work, if the CBR at 5.0 mm is larger than at 2.5 mm penetration, the test need not be redone but *both CBR values should be reported*.

BEARING RATIO TEST

Data Sheet 22

Project CBR Test Job No. Location of Project Bradley Univ. Boring No. Sample No. Description of Soil Silty Blue clay A-4(8) $w_L = 28.4\%$
 $w_p = 19.8\%$ Tested By JEB & RGL Date of Testing 6-16-76Compaction Energy: Hammer 24.5 N No. of Layers 5 Blows/layer 40 w at compaction 18.7 % Mold diam. 15.2 cm Ht. of Soil 12.7 cm Vol. 2305 cm³Wt. wet soil 4704 g Wt. dry soil, W_s 3933 g γ_{wet} 20.02 kN/m³ γ_{dry} 16.74 kN/m³Swell Data Used 5.08 cm spacer disk & adjust blows from 37 to 40

Starting time and date	Elapsed time	Mold no. <u>#1</u> Surcharge <u>44.5</u> N		Mold no. <u> </u> Surcharge <u> </u> N		Mold no. <u> </u> Surcharge <u> </u> N	
		Dial reading (x 0.01 mm)	$\% = \frac{S}{H}(100)$	Dial reading (x <u> </u>)	$\% = \frac{S}{H}(100)$	Dial reading (x <u> </u>)	$\% = \frac{S}{H}(100)$
6/16 a.m. 10:30	0 h	0	0				
	1 h	0.047	0.037%				
	2 h	0.071	0.056				
	4 h	0.108	0.085				
	14	0.140	0.110				
6/17 a.m. 10:30	24	0.190	0.150				
	37.5	0.272	0.214				
6/18 12: Noon	49.5	0.300	0.236				
12 Mid. 6/19	61.5	0.325	0.256				
12 Noon 6/20	73.5	0.335	0.264				
1:00 PM	98.5	0.355	0.280				

After Soaking

Mold no.	<u>1</u>		
Surcharge, N	<u>44.5</u>		
Initial wt. wet soil + mold + base plate	<u>12,056 g</u>		
Final wt. wet soil + mold + base plate	<u>12,130</u>		
Wt. of mold + base plate	<u>7,352</u>		
Initial wt. of wet soil, W_i	<u>4704</u>		
Wt. of water absorbed, W_w	<u>74 g</u>		
% water absorbed = W_w/W_s	<u>1.88 %</u>		

Note: Insert units in column headings as necessary.

Figure 19-6

Data from a CBR swell test.

BEARING RATIO TEST

Data Sheet 23

Project CBR Test Job No. ~Location of Project Bradley University Boring No. ~ Sample No. ~Description of Soil Silty Blue Clay A-4(8) $w_L = 28.4\%$
 $w_p = 19.8$ Tested by JEB & RGL Date of Testing 6-20-76

CBR Test Load Data (soaked, unsoaked) (Strike out as applicable.)

Penetration, mm	Mold no. <u>3</u> Surcharge <u>44.5N</u>		Mold no. <u>1</u> Surcharge <u>44.5N</u>		Mold no. <u> </u> Surcharge <u> </u>	
	Piston load dial reading (<u>$\times 0.579 \text{ kg/d}$</u>)	Load, kPa	Piston load dial reading (<u>$\times 0.579 \text{ kg/d}$</u>)	Load, kPa	Piston load dial reading (<u> </u>)	Load, kPa
0.000	0	0	0	0		
0.5	100	293	33	97		
1.0	154	452	57	167	Data plotted on Fig. 19-8 ~	
1.5	188	552	78	229		
2.0	222	651	96	282		
2.5	251	737	113	332		
3.0	277	815	131	384		
4.0	301	883	149	437		
5.0	347	1020	181	531		
6.0	388	1140	204	600		
7.5	429	1260	249	731		
9.0	494	1450	274	804		
10.0	525	1541	290	860		
12.5	635	1863	332	974		
Final water content, $w\%$ (soaked, unsoaked) sample	Top $\frac{1}{3}$	14.6%	X	Sample		
	Middle $\frac{1}{3}$	14.7		oven-dried		
	Bottom $\frac{1}{3}$	15.3		~		
	Average	14.9%		21.5%		

Wet unit wt. = 19.92 kN/m^3
20.02Dry unit wt. = 17.34 kN/m^3
16.47

No. 3

No. 1 (soaked)

Note: Insert units in column headings as necessary.

Figure 19-7 Penetration-load data for both an unsoaked and a soaked CBR test as normally run.

3. Compute the water contents and dry densities of both samples before soaking and of the final condition for the soaked sample. For the soaked sample, use data from step 18 of Procedure A.
4. Compute the percentage of swell based on the nominal initial height of the sample. Plot a curve of percent swell (ordinate) vs. elapsed time on a separate sheet of graph paper.
5. The report should compare the CBR values, present a summary of the water contents properly identified, and include the required curves. Discuss the significance of a large or small CBR value, any changes in CBR with soaking, and any swell which may occur. How might the swell be reduced (if there is a significant amount) or eliminated for this soil? Be sure your report includes classification data and the AASHTO classification of this soil.

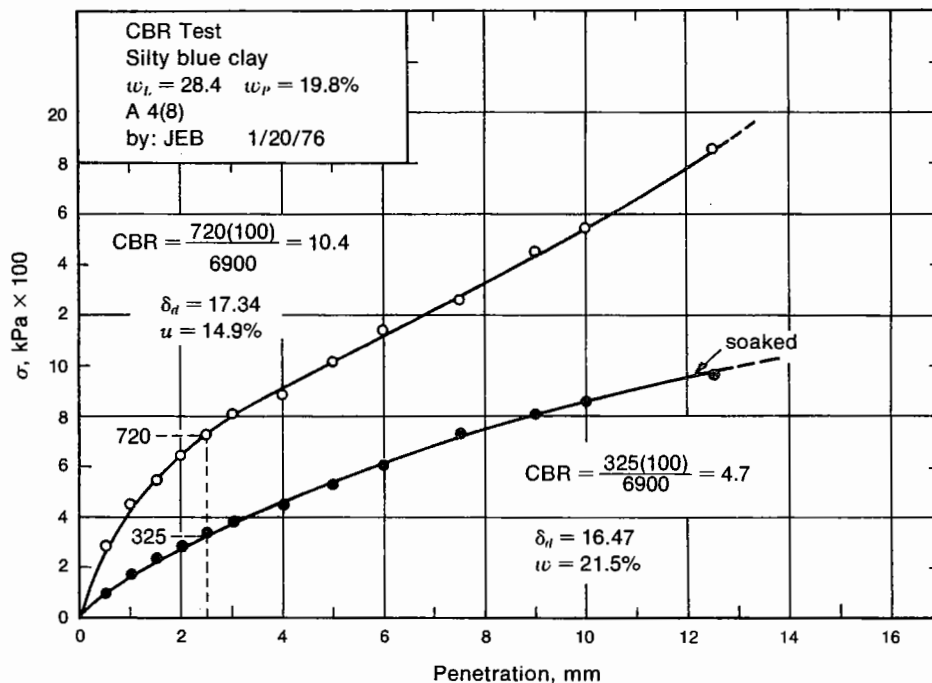


Figure 19-8

Plot of load vs. penetration to compute the CBR numbers for the soil.

Experiment No. 20

FLOW-NET CONSTRUCTION USING AN ELECTRICAL ANALOGY

References

- Herbert, R., and K. R. Rushton (1966), Ground-Water Flow Studies by Resistance Networks, *Geotechnique*, London, vol. 16, no. 1, March, pp.53-75.
- Lane, E. W., F. B. Campbell, and W. H. Price, (1934), The Flow Net and the Electric Analogy, *Civil Engineering*, October, pp. 510-514.
- Selim, M. A. (1947), Dams on Porous Media, *Transactions, ASCE*, vol. 112, pp. 488-505.
- Zanger, C. N. (1953), Theory and Problems of Water Percolation, *U.S. Bureau of Reclamation, Engineering Monograph no. 8*, April, 76 pages.

Objective

To introduce the student to the concept that the flow of a fluid through a porous medium is similar (analogous) to the flow of an electric current through a current-conducting medium.

Equipment

Vacuum-tube voltmeter (VTVM) or Wheatstone bridge (Fig. 20-1)
DC voltage supply
Teledeltos paper¹
Scissors
Silver paint to make electrodes²

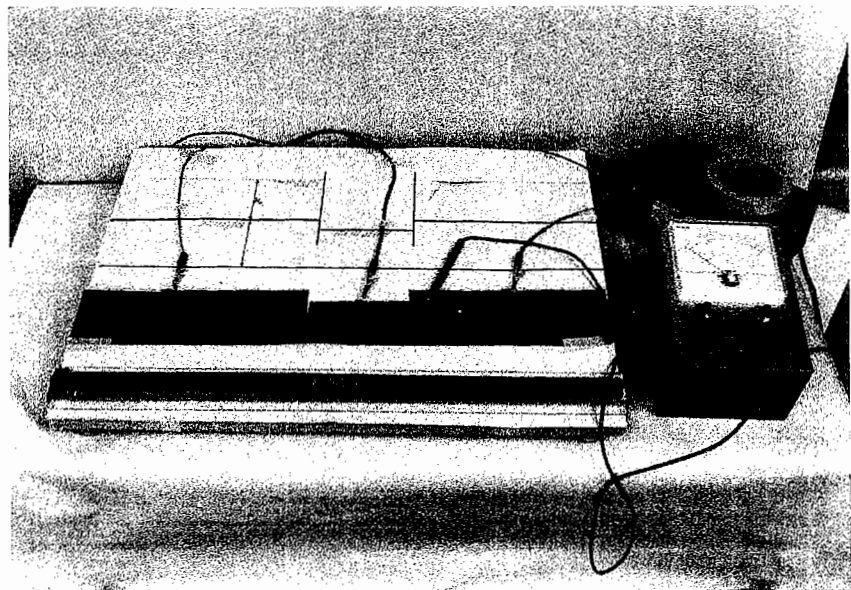


Figure 20-1

Electrical analogy setup. Cut-out in Teledeltos paper represents the conditions shown in drawing mounted above the cutout. Notches cut in paper represent sheet-pile penetration. Note that voltage is impressed from both sides to center portion.

¹Teledeltos paper can be obtained from Western Union Telegraph Co., Marketing Operation, 60 Hudson Street, New York, N.Y. 10013.

²Silver paint can be obtained from GC Electronics, Rockford, Illinois.

General Discussion

The construction of a flow net to describe the two-dimensional flow of water through a soil mass as an accurate graphical solution of the Laplace equation

$$k_x \frac{\partial^2 h}{\partial x^2} + k_y \frac{\partial^2 h}{\partial y^2} = 0 \quad (20-1)$$

is a time-consuming operation. The problem becomes even more difficult if the coefficients of permeability (k_x and k_y) in Eq. (20-1) are different in the x and y directions. The problem becomes very difficult if the soil is stratified or if the boundary conditions are irregular.

It has been found that the flow of an electric current from a high to a low potential is analogous to the flow of water caused by a differential head. From this similarity it follows that one can construct the shape of the porous soil mass out of an electrical conducting material and impress a differential voltage at locations to simulate the differential head of water between the two (or more) points. The voltages, as obtained with an electrical probe, at various locations within the model will be to some scale the potential water head at that point. Electrical conducting models can be constructed of:

1. Wire mesh (as used for the No. 200 sieve or other fine mesh).
2. Metal plates or using metal foil.
3. Fine sand which has been treated to become electrically nonconductive, built into a model, then saturated with a water solution containing an electrolyte. This may require using ac voltage to avoid polarizing parts of the model when the probe is used.
4. Teledeltos paper (or any other electrically conducting paper of commercial or laboratory manufacture).

Other electric analogy model materials may be used but items 1 and 4 above are probably the most common.

Note that it is possible to build a model for both two-dimensional (either plan or elevation) flow and three-dimensional flow.

Different coefficients of permeability, as for stratification or lenses of different materials, can be simulated in a model by inserting pieces of material of greater or less conductivity depending on the user's ingenuity and perseverance.

Several simple examples using Teledeltos (same as using wire screen) paper cutouts will be illustrated in the scope of this text. In Fig. 20-2a, the existing soil mass is shown on an impervious base (a boundary condition). Figure 20-2b gives the corresponding cutout and locations of the impressed voltage to simulate the head of water shown in Fig. 20-2a.

The VTVM is set to read an impressed voltage between electrodes A and B of the cutout of, say, 10 volts (V) by an appropriate adjustment of the dc power supply.¹ This is analogous to the potential water head of 13.5 shown in Fig. 20-2a; thus each 1.35 m of water head corresponds to a differential voltage of 1 V on the VTVM scale. It is possible to use a one-to-one voltage-to-water head analogy, but this may require voltages high enough to be uncomfortable to work with or to cause sparking—even with the very low amperages which the dc power supply should output. The current supplied by the power source should be in the milliamperage range.

Now, to find an equipotential line (line of equal total head), it is necessary to take a probe from the "low" side of the VTVM and find a location, such as C in the

¹With Teledeltos paper you can use dc voltage.

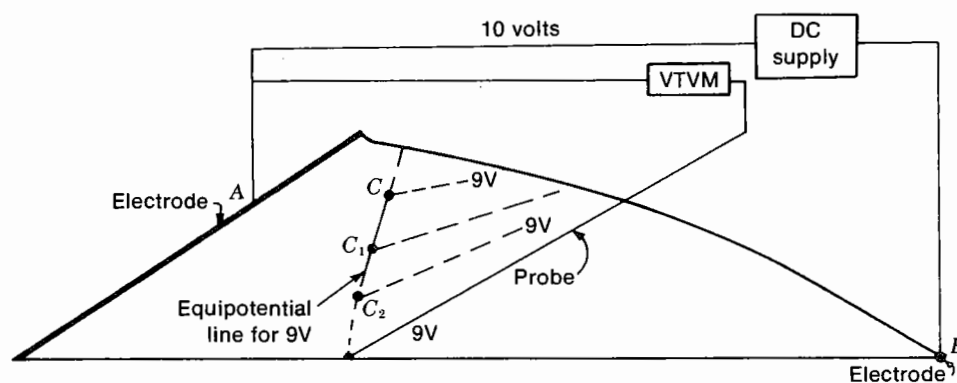
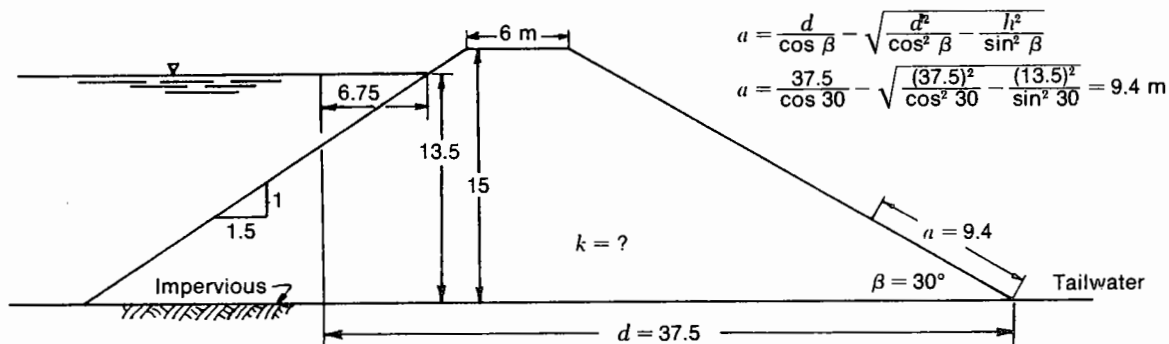


Figure 20-2
Pervious earth dam on an impervious base.

cutout of Fig. 20-2b, which corresponds to, say, 9 V (we started with 10 V in this example). This corresponds to the remaining head potential in the soil mass of

$$\frac{h_L}{1} V = 13.5 \text{ ft of } \frac{H_2O}{10} V$$

from which

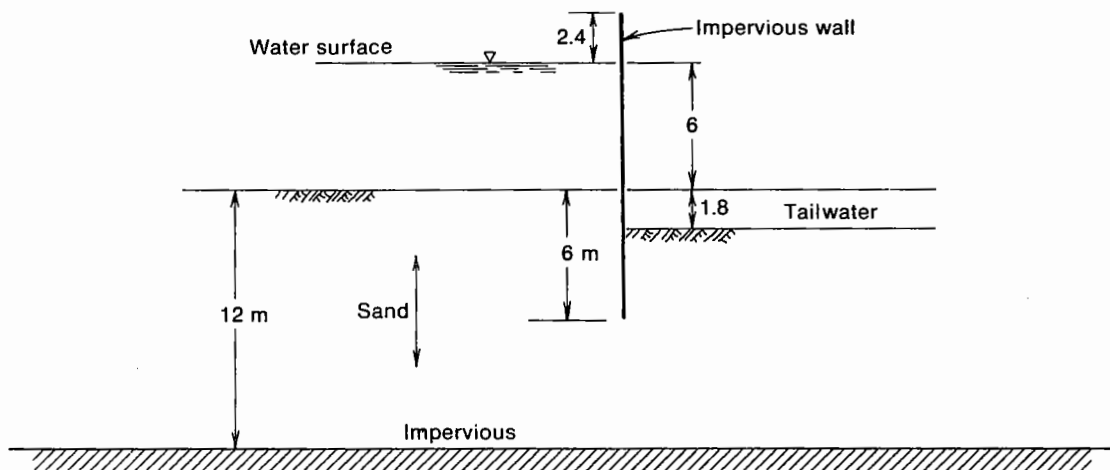
$$h_L \frac{13.5}{10} = 1.35 \text{ m}$$

Thus the remaining head is

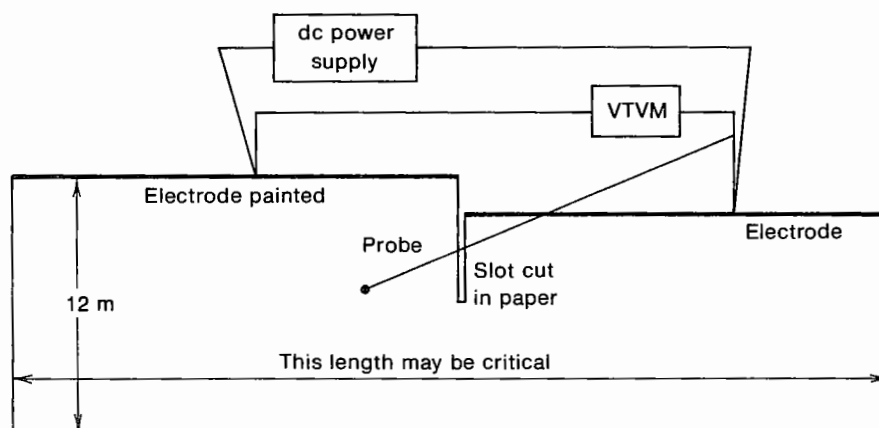
$$h_{\text{remaining}} = 1.35 \times 9 \text{ V} = 12.15 \text{ m of water}$$

If one finds a series of points such as C for which the voltmeter reads 9 V at each location, the locus of points so established is an equipotential line for a remaining water head of 12.15 m of water. A similar analysis for 8, 7, 6, etc., can be made, yielding equipotential lines for 10.8, 9.45, 8.1 m, etc., remaining water head.

It is difficult to obtain the flow-path boundaries to complete the flow net using this technique, but with the equipotential lines fairly accurately drawn, the sketching in of flow paths to make squares to satisfy Eq. (20-1) is not very difficult.



(a) The field conditions.



(b) The cut-out made of Teledeltos paper with electrodes painted in proper location together with impressed voltage and VTVM.

Figure 20-3

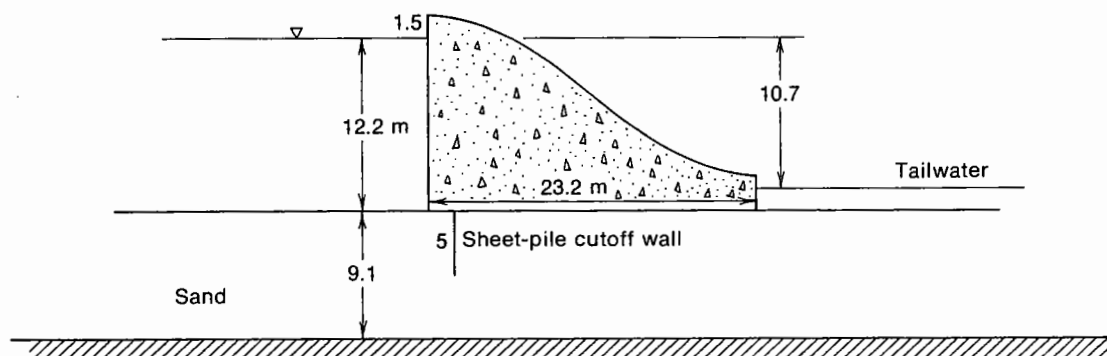
Single sheet-pile wall for flow-net analysis.

Figure 20-3a and b shows an existing sheet-pile wall system and the corresponding cutout for use in the electrical-analogy solution to establish equipotential lines.

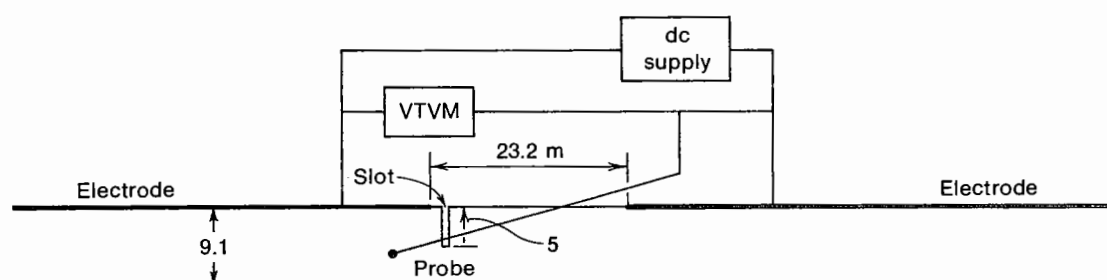
In Fig. 20-3b, the thin slot is an electric barrier simulating the impervious sheet-pile wall. As before, the impressed voltage from A to B simulates the differential head of water (of 6 m in this case). Notice that the effect of tailwater is to establish the differential head h across the seepage path of the system. Again, by probing at a particular voltage, one establishes a locus of points for an equipotential line. Flow paths should be sketched in by eye using the concept of establishing squares to complete the flow net for the system.

For problems of the type shown in Figs. 20-3 and 20-4, the upstream and downstream boundary conditions (large distance of $L \rightarrow \infty$) can be approximated by using a model length three to six times the height H or stratum thickness. The proper length can be checked by observing how the equipotential lines intersect into the bottom boundary. That is, if the equipotential lines do not intersect off a smooth curve and at right angles, the model length is not adequate for the thickness of the pervious soil stratum.

If the sheet-pile wall is pervious, the situation may be simulated by using a



(a) Field conditions.



(b) The cut-out made from Teledeltos paper with electrodes painted in proper location together with impressed voltage and VTVM.

Figure 20-4
Concrete dam on pervious foundation with sheet-pile cutoff wall.

slotted rather than a full-slot wall cutout. If the soil is stratified, or changes in coefficient of permeability k , this may be simulated by increasing or decreasing the conductance of the paper by adding (add a light coat of paint) or removing (punch holes) electrolyte from the cutout in the appropriate zone(s). These techniques are "trial and adjustment" but may be used to indicate possible ranges of expected water flow—especially in view of the fact that considerable uncertainty exists in determining the coefficient of permeability of a soil.

Teledeltos paper is an excellent material for use as an electrical-analogy cutout. It is a paper with a graphite coating, and it has been found that the conductivity may vary slightly between x and y directions. However, for the solution of most soil hydraulic problems, any differences in conductivity are of academic importance. This is especially so when one considers the degree of precision associated with the determination of the coefficient of permeability of the soil.

In order to simulate water in contact at more than a point at any boundary, it is necessary to paint an electrode of appropriate length on the paper cutout so that the electric current is simulating the water on entering and exiting the soil mass. The electrodes should be highly conductive with respect to the paper, and silver paint (silver base) is recommended for this purpose. One might clamp metal strips to the paper cutout, but this procedure usually results in an awkward model to work with and the silver paint is preferred. The electrodes should be painted as straight as possible and should use a relatively narrow strip because of the reduction factor already involved in the small-size model with respect to the field simulation.

The Teledeltos paper will possess erratic conducting qualities if it becomes perforated or is creased or crumpled. Therefore, it should be carefully handled and stored.

If $k_x \neq k_y$, from flow-net theory it is recalled that the x or y dimensions can be modified using the following transformations:

$$x' = x \sqrt{\frac{k_y}{k_x}}$$

or

$$y' = y \sqrt{\frac{k_x}{k_y}}$$

These are made so that when using the transformations, squares can be sketched when constructing the flow net. For the electrical-analogy scale cutout, the dimensions should also be adjusted, using the appropriate transformation equation from above prior to cutting the mode.

Procedure

1. Each student will make a scale cutout using Fig. 20-1p, 20-2p, or 20-3p, as designated by the instructor, with the Teledeltos paper furnished.
2. Carefully paint the electrodes on the cutout using the electrode paint furnished. Use electrode locations which are appropriate to determine equipotential lines.
3. Impress a convenient voltage across the electrodes of the cutout and, with the probe from the VTVM, locate at least 10 equipotential lines on the model. Lightly mark the locus of points by circling them with a pencil, *being very careful not to perforate the paper cutout*. Remember that holes will change the conductivity (coefficient of permeability) of the electrical model.
4. Make a tracing of the cutout and equipotential line locations and complete the flow-net construction by freehand sketching of the flow paths. Compute the seepage quantity per meter of structure per day.
5. Redo the assigned problem of step 1 above using freehand work for both equipotential and flow-path lines. Do not be highly concerned with making an accurate drawing; merely make a sketch (to scale, of course) which does not "look too bad" and compute the seepage per meter of structure per day as in step 4.
6. Compare the results of steps 4 and 5 and comment on any significant differences. In your report be sure to include your paper cutout mounted on a sheet of heavy paper or cardboard so that it does not get torn or lost.

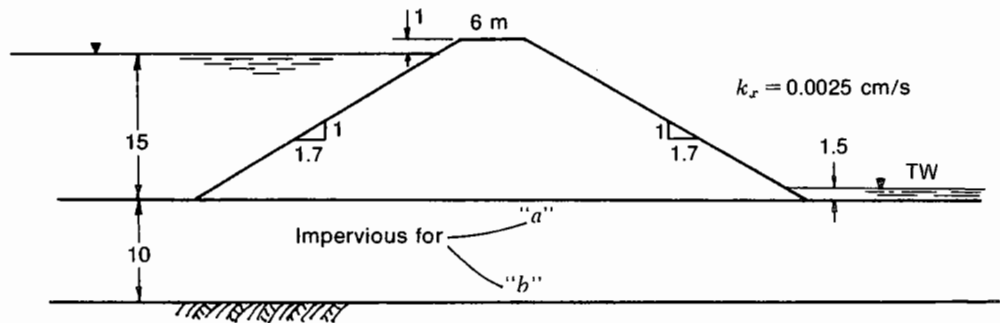


Figure 20-1p

Not to scale.

- (a) Find the seepage quantity in $\text{m}^3/\text{m}/\text{day}$ for the structure shown if it is on an *impervious* base. (b) Find the seepage quantity in $\text{m}^3/\text{m}/\text{day}$ for the structure shown if it is on a pervious base and both dam and base have the same coefficients of permeability. (c) Redo (a) if $k_y = 0.25k_x$, where k_x is the value shown in Fig. 20-1p.

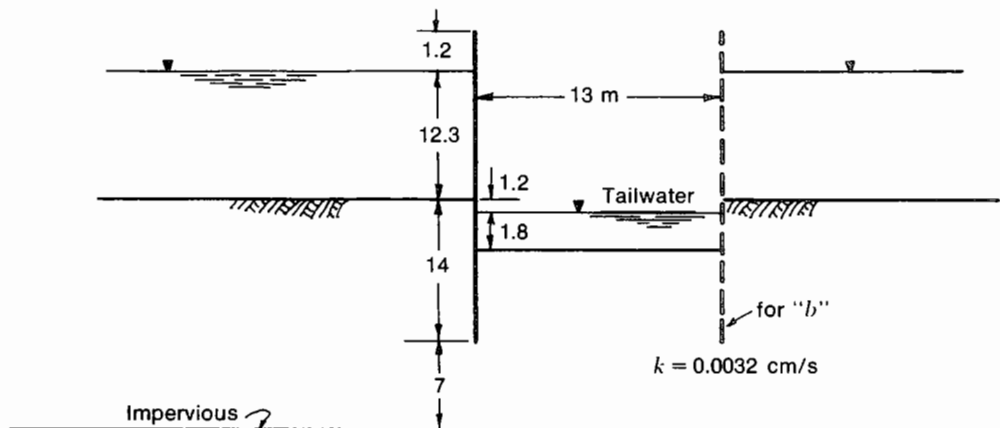


Figure 20-2p

Not to scale.

- (a) Find the seepage quantity in $\text{m}^3/\text{m}/\text{day}$ for the sheet-pile wall on the left only. (b) Find the seepage quantity in $\text{m}^3/\text{m}/\text{day}$ for a cofferdam situation above if both sheet-pile walls exist.

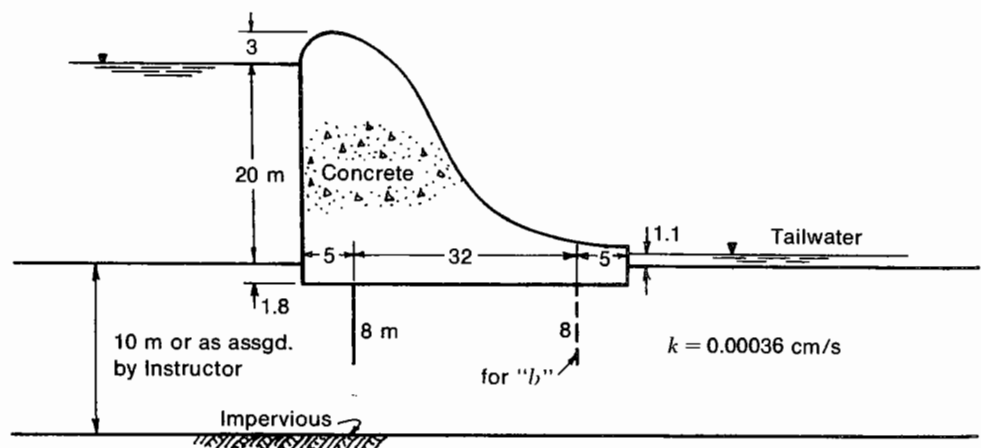


Figure 20-3p

Not to scale.

- (a) Find the seepage in $\text{m}^3/\text{m}/\text{day}$ of dam as shown for the sheet-pile cutoff wall at the *upstream* location (heavy line). (b) Find the seepage in $\text{m}^3/\text{m}/\text{day}$ of dam as shown for the sheet-pile cutoff wall at the *downstream* location (dashed line). (c) Find the seepage in $\text{m}^3/\text{m}/\text{day}$ of dam as shown for the sheet-pile cutoff walls at *both* locations.

Experiment No. 21

VOLUMETRIC—GRAVIMETRIC RELATIONSHIPS

References

Archimedes' principle that a body submerged in a liquid will displace its own volume as found in any physics textbook

Objective

To introduce the student to the concept of dry and saturated unit weights, void ratio, soil structure (state), and a method of computing the specific gravity of a coarse-grained material.

Equipment

Large straight-sided container¹ of known volume
Steel straightedge
Quantity of oven-dry sandy gravel (say, 20 to 2 mm)
Balance
Graduated cylinder (500 to 1000 ml)

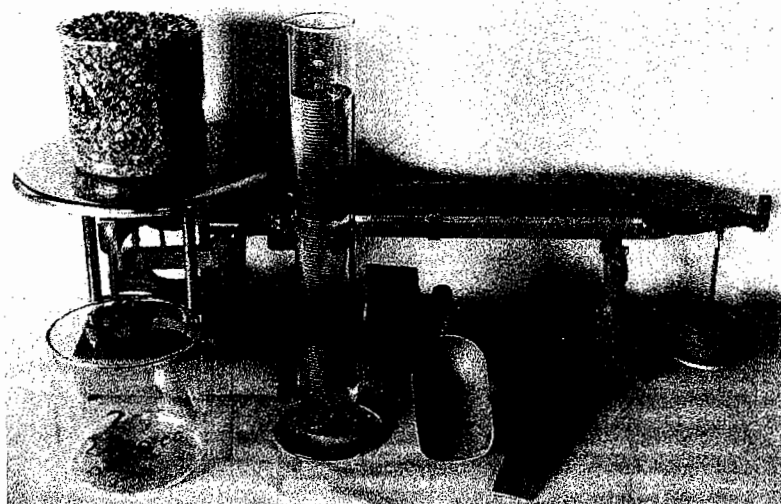


Figure 21-1

Equipment for volumetric-gravimetric relationships. Filled battery jar on 20-kg balances, 1000-ml graduated cylinder with water to pour into gravel in battery jar after initial weighing for dry unit weight. Note volume of jars already determined and jar marked to save time.

Procedure

1. Each group carefully weigh the empty volume container and fill with soil, carefully strike the surface flush, and weigh. If the volume of soil changes due to weighing vibrations, you have introduced a state (soil structure) change. Add additional soil, reweigh, and/or repeat as necessary until weighing vibrations does not cause further state change. Record the final weight of soil + container on your data sheet in the space provided.

¹The author suggests using a battery jar of dimensions 10-cm diam \times 15-cm height (vol approx. 1200 cm³) or, preferably, 15-cm diam \times 15-cm height (vol approx. 2400 cm³) and readily available from laboratory supply companies. Also round kitchen coffee or sugar containers can be used if made of transparent plastic so that observation can be made of soil state and entrapped air bubbles.

2. Fill the graduated cylinder with temperature-stabilized (approximately 20°C) tap water (let tap run sufficiently that cold water is provided). Carefully pour the water into the container of soil, by pouring around the edges and slowly, so as to avoid as much as possible any entrapment of air in the voids in the soil. Fill the volume container level full with water but do not overflow so that no water is lost. Also be careful to fill level full and not "rounded" due to meniscus action. Observe the sides of the container as you fill it and if air bubbles appear to be trapped, gently rock the container or lightly tap the container at the bubble to dislodge it. Be very careful not to introduce a soil-state change at this point.

Record the volume of water used to fill the soil container.

3. Carefully place the filled container on the balance (you may wish to place the container partially filled with water on the balance, then finish filling, being careful to keep the container level so that it is completely filled with water at the end of the operation.)

Record the weight of soil + water + container. Compare the difference in weight with the milliliters of water added. The difference should only be 2–5 grams; if more than this, recheck the graduated cylinder and/or weighing.

Computations

1. Compute the dry density as

$$\gamma_d = \frac{\text{weight of dry soil}}{\text{volume of container}} \quad \text{g/cm}^3 \quad (21-1)$$

$$\gamma_d = \text{g/cm}^3 \times 9.807 \quad \text{kN/m}^3 \quad (21-2)$$

2. Compute the saturated unit weight as

$$\gamma_{\text{sat}} = \frac{\text{weight of saturated soil}}{\text{volume of container}} \quad \text{g/cm}^3 \quad (21-3)$$

$$\gamma_{\text{sat}} = \text{g/cm}^3 \times 9.807 \quad \text{kN/m}^3 \quad (21-4)$$

3. Compute the void ratio as follows:

$$\text{Compute } V_s = \text{vol. container} - \text{vol. of H}_2\text{O} \quad \text{cm}^3$$

$$\text{Compute the volume of voids, } V_v = \text{vol. of H}_2\text{O} \quad (\text{ml} \cong \text{cm}^3)$$

Compute the void ratio e as

$$e = \frac{V_v}{V_s} \quad (21-5)$$

4. Compute the specific gravity of the soil solids G_s as

$$G_s = \frac{W_s}{V_s \gamma_w} \quad (21-6)$$

Refer to Experiment No. 7 for typical values of G_s and compare to your value.

The Report

1. Prepare a neat line sketch (phase diagram) showing all volumes and weights.
2. Comment on the limitations of computing G_s in this manner. Why has the soil size been limited?
3. Comment on any improvements which can be made in the test, such as: Would it be better to use a few drops of wetting agent in the water to reduce air-bubble formation on soil or container sides? How does the size of the volume container affect results? What is the error introduced in not using distilled and/or deaired water at 4°C instead of tap water at about 20°C?
4. Derive the constant 9.807 used to convert g/cm^3 to kN/m^3 .
5. Put the above information in a folder with a cover page and submit as assigned.

VOLUMETRIC GRAVIMETRIC RELATIONSHIPS

Data Sheet 24

Project _____

Location of Project _____

Description of Soil _____

Tested by _____ Date _____

Volume of container, $V_c =$ _____ cm^3

Weight of dry soil + container = _____ g

Weight of soil + water + container = _____ g

Weight of container = _____ g

Weight of dry soil, $W_s =$ _____ g

Weight of saturated soil, $W_t =$ _____ g

Volume of water to saturate soil, $V_v =$ _____ ml

Compute:

$$V_s = V_c - V_v = \text{_____} = \text{_____} = \text{_____} \text{ cm}^3$$

$$\gamma_d = \text{_____} = \text{_____} \text{ g/cm}^3 \times 9.807 = \text{_____} \text{ kN/m}^3$$

$$\gamma_{\text{sat}} = \text{_____} = \text{_____} \text{ g/cm}^3 \times 9.807 = \text{_____} \text{ kN/m}^3$$

$$e = \text{_____} = \text{_____}$$

$$G_s = \text{_____} = \text{_____}$$

Experiment No. 22

UNIT WEIGHT OF COHESIVE SOILS

References

ASTM D2937-71

Objective

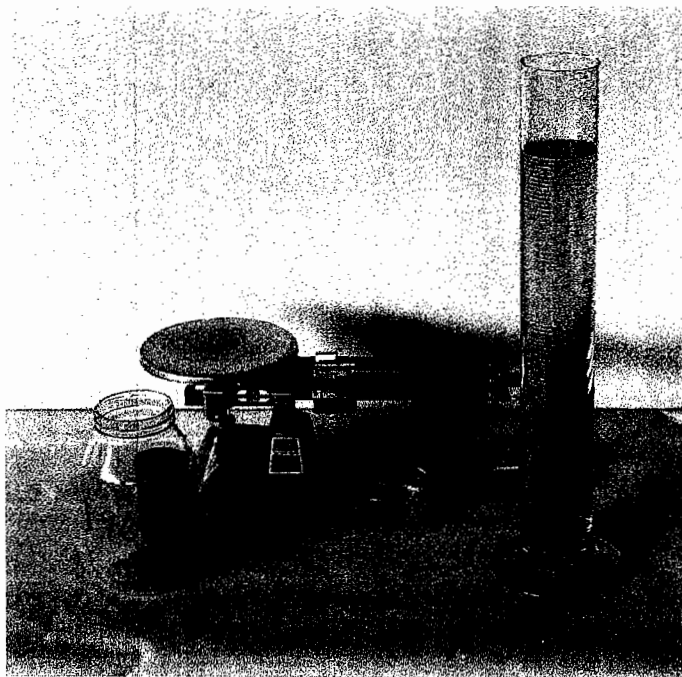
Introduce the student to a rapid method of finding the unit weight of a cohesive soil.

Equipment

Container of known volume (such as a 1-quart (or 1-liter) canning jar or a discarded mayonnaise jar)
Balance weighing to 0.1 g
Graduated cylinder (500 ml or larger)

Figure 22-1

Equipment for a simple and rapid determination of unit weight of cohesive soil. Volume of glass jar shown is 969 cm³. Small balances shown will be adequate (usually) for weight determinations. Use a graduated cylinder of no less than 500 ml for this size volumetric jar (1000 ml shown). Use soil sample large enough that least count of graduated cylinder does not materially affect results (or use differences in weights for volume of water).



General

This procedure can be used to obtain the displaced volume of any cohesive soil that will not slake apart in water for the 1 to 2 min required to perform the test. This procedure is not applicable for porous materials or very dry soils unless they are very dense as any appreciable water adsorption via capillary action or otherwise will affect the results. For a very large number of soils this procedure will be quite adequate, rapid, and economical. This procedure is applicable for soils located at a depth in the ground; for shallow soils the methods of Experiment No. 10 may be used.

Procedure

1. Determine the volume of the volumetric container as V_c .
2. Obtain a tube sample (if available) of cohesive soil.¹ Obtain a specimen which does not contain any obvious holes.
3. Carefully weigh the sample to obtain W_t .

For best results the test samples should weigh 400 to 700 g. Place the sample in the volumetric container without fracturing it.

4. Fill the graduated cylinder with temperature-stabilized tap water (approx. 20°C by letting tap run for a period of time). Then *quickly* fill the volumetric container containing the soil sample and record the volume of water used as V_w . Quickly empty the volumetric container, remove the soil sample and blot surface dry with paper towels and reweigh. If the initial and final weights are within 1 or 2 g, the test is quite satisfactory—more than this will require judgment of test acceptance.

Compute the volume of sample as

$$V_s = V_c - V_w$$

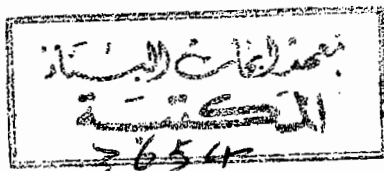
5. Place the wet soil in a dish and oven-dry overnight to obtain W_s (unless the water content is known).
6. Compute the wet and dry densities of the soil as follows:

$$\gamma_{\text{wet}} = \frac{W_t}{V_s} \times 9.807 \quad \text{kN/m}^3 \quad (22-1)$$

$$\gamma_{\text{dry}} = \frac{W_s}{V_s} \times 9.807 \quad \text{kN/m}^3 \quad (22-2)$$

The Report

1. Complete the data sheet and show densities.
2. Comment on test limitations such as
 - a. When test is not applicable
 - b. Would some other liquid be better to use than water
 - c. Size of sample and/or alternative sample preparation (grease coating, some other coating, etc.)
3. Put project in a folder with a cover sheet and submit as required.



1980-81 ٢

¹Where tube samples are not readily available (student soil labs), the author suggests a set of samples be made up prior to the laboratory period using the Harvard Miniature Compaction Device (see Experiment No. 9) or some modification of it to produce samples of known volume and density.

UNIT WEIGHT OF A COHESIVE SOIL

Data Sheet 25

Project _____ Job No. _____

Location of Project _____

Description of Soil _____

Tested by _____ Date _____

Boring No. _____ Boring No. _____

Depth _____ Depth _____

Wt. of sample, W_t = _____ gWt. of sample, W_t = _____ gVol. of container, V_c = _____ cm^3 Vol. of container, V_c = _____ cm^3

Initial grad. read. = _____ ml

Initial grad. read. = _____ ml

Final grad. read. = _____ ml

Final grad. read. = _____ ml

Volume of water, V_w = _____ mlVolume of water, V_w = _____ mlVolume of soil, V_s = _____ cm^3 Volume of soil, V_s = _____ cm^3

Wet density:

$$\gamma_{\text{wet}} = W_t/V_s = \text{_____ g/cm}^3$$

Wet density:

$$\gamma_{\text{wet}} = W_t/V_s = \text{_____ g/cm}^3$$

$$\gamma_{\text{wet}} = \text{g/cm}^3 \times 9.807 = \text{_____ kN/m}^3$$

$$\gamma_{\text{wet}} = \text{gm/cm}^3 \times 9.807 = \text{_____ kN/m}^3$$

*Computation of Dry Unit Weight*Water content, $w\%$ = _____Water content, $w\%$ = _____

$$\gamma_{\text{dry}} = \frac{\gamma_{\text{wet}}}{1 + w} = \text{_____ kN/m}^3$$

$$\gamma_{\text{dry}} = \frac{\gamma_{\text{wet}}}{1 + w} = \text{_____ kN/m}^3$$

$$\gamma_{\text{dry}} = W_s/V_s = \text{_____ kN/m}^3$$

$$\gamma_{\text{dry}} = W_s/V_s = \text{_____ kN/m}^3$$

WATER CONTENT DETERMINATION

Data Sheet 1

Project _____ Job No. _____

Location of Project _____

Description of Soil _____

Tested By _____ Date of Testing _____

Date of Weighing _____

Boring no.					
Container no. (cup)					
Wt. of cup + wet soil					
Wt. of cup + dry soil					
Wt. of cup					
Wt. of dry soil					
Wt. of water					
Water content, $w\%$					

Boring no.					
Container no. (cup)					
Wt. of cup + wet soil					
Wt. of cup + dry soil					
Wt. of cup					
Wt. of dry soil					
Wt. of water					
Water content, $w\%$					

[illegible]

4

5

6

7

8

9

10

ATTERBERG LIMITS DETERMINATION

Data Sheet 3

Project _____ Job No. _____

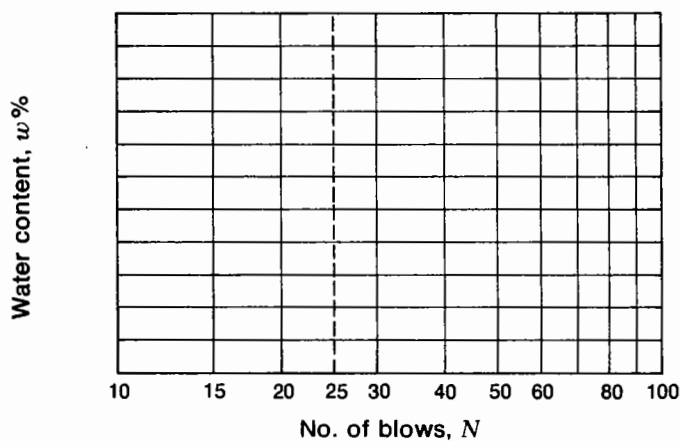
Location of Project _____ Boring No. _____ Sample No. _____

Description of Soil _____

Depth of Sample _____ Tested By _____ Date _____

Liquid Limit Determination

Can no.						
Wt. of wet soil + can						
Wt. of dry soil + can						
Wt. of can						
Wt. of dry soil						
Wt. of moisture						
Water content, $w\%$						
No. of blows, N						



Flow index $F_i =$ _____

Liquid limit = _____

Plastic limit = _____

Plasticity index $I_p =$ _____

Plastic Limit Determination

Can no.				
Wt. of wet soil + can				
Wt. of dry soil + can				
Wt. of can				
Wt. of dry soil				
Wt. of moisture				
Water content, $w\% = w_p$				

SHRINKAGE LIMIT TEST

Data Sheet 4

Project _____ Job No. _____

Location of Project _____ Boring No. _____ Sample No. _____

Description of Soil _____ Depth of Sample _____

Tested By _____ Date of Testing _____

Wt. of coated dish + wet soil = _____ g

Wt. of coated dish + dry soil = _____ g

Wt. of coated dish = _____ g

Wt. of soil, W_s = _____ gWt. of Water, W_w = _____ gWater content, $w_o\%$ = _____Vol. of Wet Soil, V_o = _____ cm^3 (Step 5 of procedure: Vol. of shrinkage dish = V_o).Vol. of Dry Soil, V_f = _____ cm^3 (Step 7 of procedure)Shrinkage limit, $w_s = w_o - \frac{(V_o - V_f) \gamma_w}{W_s} \times 100 =$ _____Shrinkage ratio, $SR = W_s/V_f =$ _____

Data Sheet 5

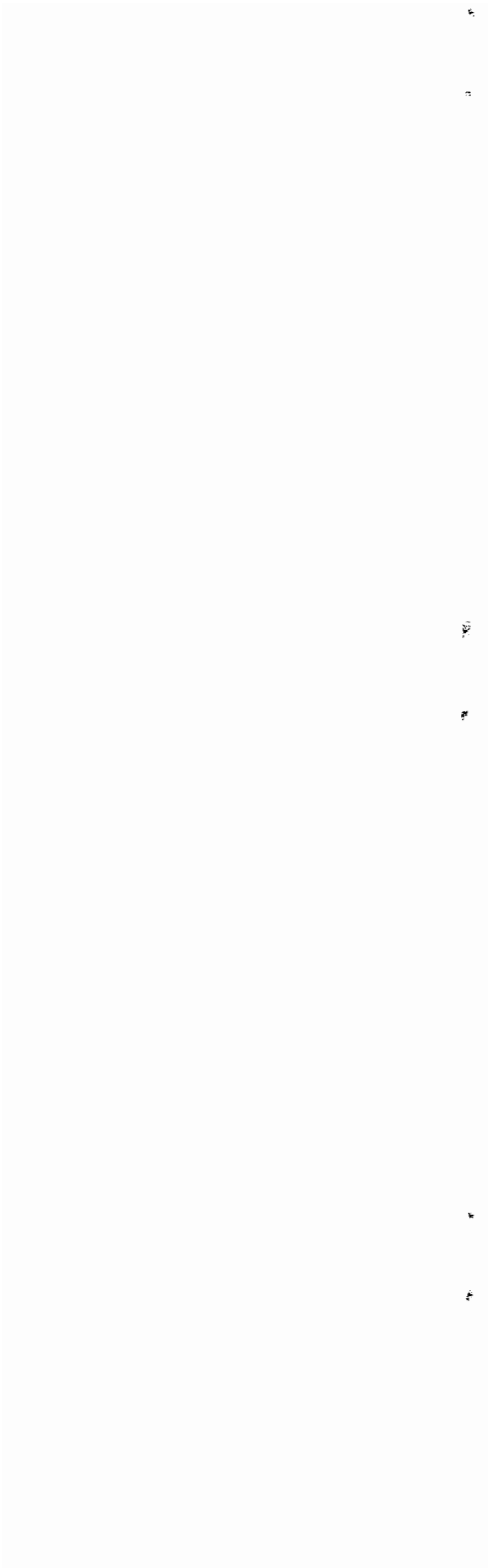
Tested By _____ Date of testing _____

Nominal diameter of largest particle	Approximate minimum Wt. of sample, g
No. 10 sieve	200
No. 4 sieve	500
3/4 in.	1500

Wt. of dry sample + container	
Wt. of container	
Wt. of dry sample, W_s	

[illegible]

$$\% \text{ passing} = 100 - \sum \% \text{ retained.}$$



GRAIN SIZE DISTRIBUTION

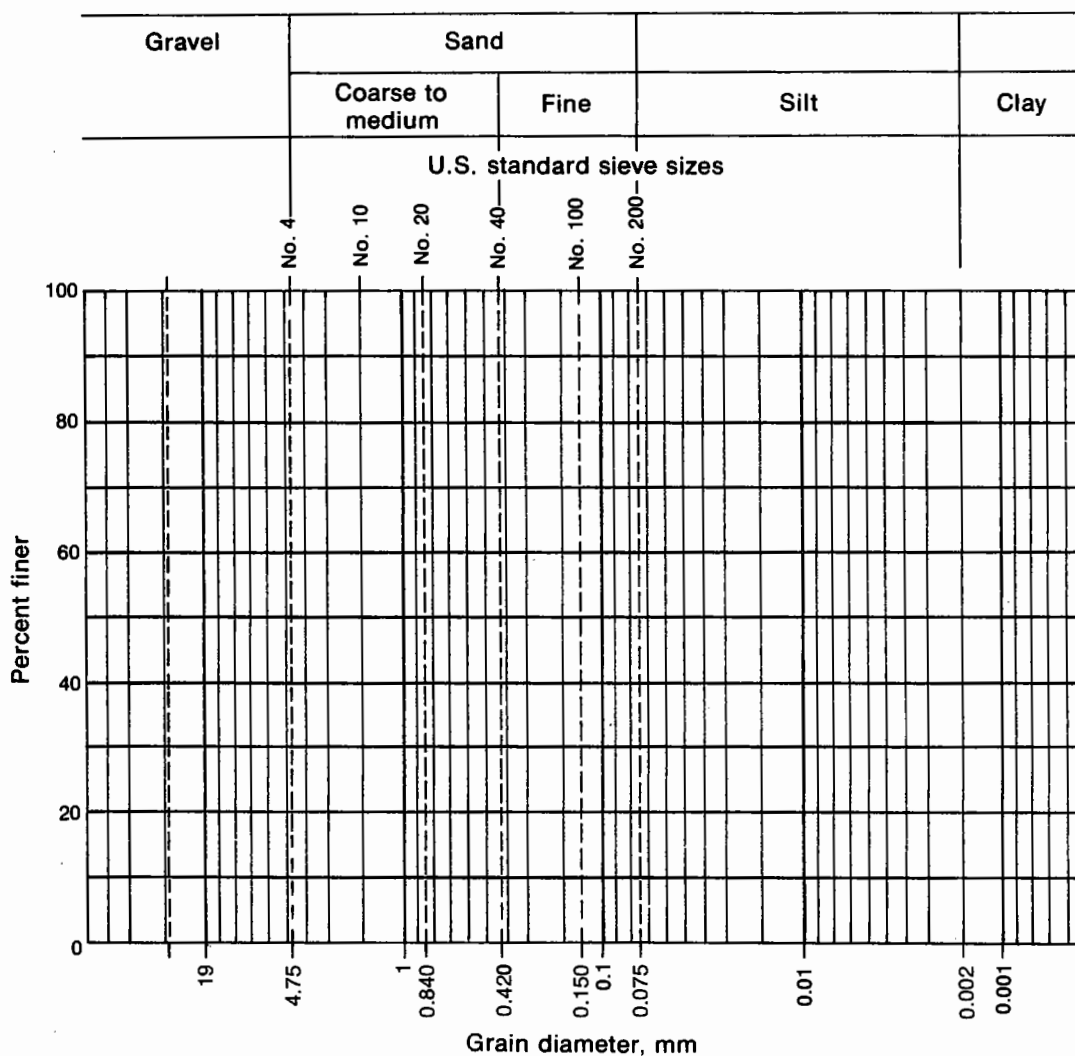
Data Sheet 6

Project _____ Job. No. _____

Location of Project _____ Boring No. _____ Sample No. _____

Description of Soil _____ Depth of Sample _____

Tested By. _____ Date of Testing _____



Visual soil description _____

Soil classification:

_____ System _____

2

3

4

5

6

7

Data Sheet 7

Project _____ Job No. _____

Location of Project _____ Boring No. _____ Sample No. _____

Description of Soil _____ Depth of Sample _____

Tested By _____ Date of Testing _____

Hydrometer analysis

Hydrometer no. _____ G_s of solids = _____ u = _____Dispersing agent _____ Amount _____ Wt. of soil, W_s _____

Zero correction _____ Meniscus correction _____

[illegible]

$$R_c = R_{\text{actual}} - \text{zero correction} + C_T$$

$$\% \text{ finer} = R_c(u)/W_s$$

$$D = K\sqrt{L/t}$$

SPECIFIC GRAVITY OF SOIL SOLIDS (G_s)

Data Sheet 8

Project _____ Job No. _____

Location of Project _____ Boring No. _____ Sample No. _____

Description of Soil _____ Depth of Sample _____

Tested By _____ Date of Testing _____

Test no.				
Vol. of flask at 20°C				
Method of air removal ^a				
Wt. flask + water + soil = W_{bws}				
Temperature, °C				
Wt. flask + water ^b = W_{bw}				
Evap. dish no.				
Wt. evap. dish + dry soil				
Wt. of evap. dish				
Wt. of dry soil = W_s				
$W_w = W_s + W_{bw} - W_{bws}$				
$G_s = \alpha W_s / W_w$				

^aIndicate vacuum or aspirator for air removal.

^b W_{bw} is the weight of the flask filled with water at same temp. $\pm 1^\circ\text{C}$ as for W_{bws} or value from calibration curve at T of W_{bws} .

Remarks _____

Average specific gravity of soil solids (G_s) = _____

SOIL CLASSIFICATION

Data Sheet 9

Project _____

Location of Project _____

Tested By _____ Date _____

Soil Identification (Container No., etc.) _____

Sieve Analysis (washed, dry)

Sieve	% Passing	Index Properties
No. 4	_____	$w_L =$ _____
No. 10	_____	$w_P =$ _____
	_____	$I_P =$ _____
No. 40	_____	_____
	_____	$C_U =$ _____
No. 100	_____	$C_C =$ _____
No. 200	_____	

Classification (write descript.) _____

Symbols: AASHTO: _____ Unified: _____

Soil Identification (Container No., etc.) _____

Sieve Analysis (washed, dry)

Sieve	% Passing	Index Properties
No. 4	_____	$w_L =$ _____
No. 10	_____	$w_P =$ _____
	_____	$I_P =$ _____
No. 40	_____	_____
	_____	$C_U =$ _____
No. 100	_____	$C_C =$ _____
No. 200	_____	

Classification (write descript.) _____

Symbols: AASHTO: _____ Unified: _____

Data Sheet 10

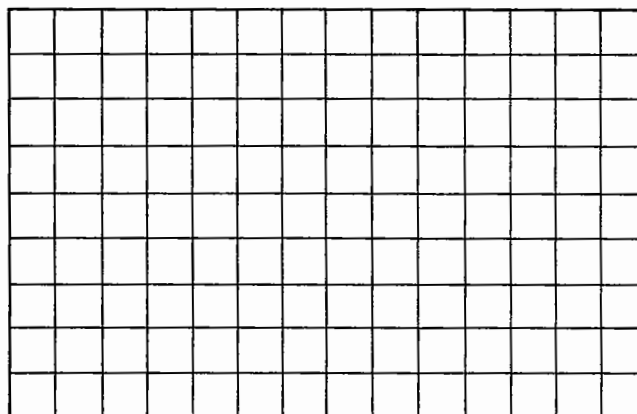
Location of Project _____ Boring No. _____ Sample No. _____

Test Performed By _____ Date of Test _____

Mold dimensions: Diam. _____ cm Ht. _____ cm Vol. _____ cm³

[illegible]

Assumed water content						
Water content, w %						
Wt. of soil + mold						
Wt. of mold						
Wt. of soil in mold, g						
Wet density, kN/m^3						
Dry density γ , kN/m^3						

Dry density γ_{dry} , kN/m^3 

Optimum moisture = _____% Maximum dry density = _____ kN/m³

FIELD DENSITY TEST (Sand cone, Balloon)

Data sheet 11

Project _____ Job. No. _____

Location of Project _____

Description of Soil _____

Test Performed By _____ Date of Test _____

Laboratory Data from Field Test

Sand-cone method

Wt. of wet soil + can _____

Wt. of can _____

Wt. of wet soil, W' _____

Wt. of wet soil + pan _____

Wt. of dry soil + pan _____

Wt. of pan _____

Wt. of dry soil _____

Water content, $w\%$ _____

Balloon method

Wt. of wet soil + can _____

Wt. of can _____

Wt. of wet soil, W' _____

Wt. of wet soil + pan _____

Wt. of dry soil + pan _____

Wt. of pan _____

Wt. of dry soil _____

Water content, $w\%$ _____*Field Data*

Sand-cone method

Type of sand used _____

Unit wt. of sand, $\gamma_{\text{sand}} =$ _____ g/cm³

Wt. of jug + cone before use _____ g

Wt. of jug + cone after use _____ g

Wt. of sand used (hole + cone) _____ g

Wt. of sand in cone (from calibration) _____ g

Wt. of sand in hole, W _____ gVol. of hole, $V_h = W/\gamma_{\text{sand}} =$ _____ cm³

Balloon method

Correction factor CF = _____

Final scale reading _____ cm³Initial scale reading _____ cm³Vol. of hole, V'_h _____ cm³Vol. of hole = V'_h (CF) _____ cm³*Density of Soil*Wet density $\gamma_{\text{wet}} = W'/V_h =$ _____ kN/m³ Dry density $\gamma_{\text{dry}} = \gamma_{\text{wet}}/(1 + w) =$ _____ kN/m³

FIELD DENSITY TEST

Data Sheet 12

Name _____ Date of Testing _____

*Calibration Data*I. *Sand-cone method*

A. Sand density determination

Sand used _____

Type of vol. measure _____ Vol., V_m _____ cm^3

Wt. of sand to fill volume measure: Trial no. 1 _____

Trial no. 2 _____

Trial no. 3 _____

Average wt W_a _____Density of sand, $\gamma_{\text{sand}} = W_a/V_m =$ _____ g/cm^3

B. Wt. of sand to fill cone

Wt. of filled jug + cone = _____

Wt. after trial No. 1 = _____ Wt. used = _____

Wt. after trial No. 2 = _____ Wt. used = _____

Wt. after trial No. 3 = _____ Wt. used = _____

Average wt. to fill cone = _____ g

II. *Volume measure (balloon apparatus) calibration*

Type of container used _____

Vol. of container, $V_c =$ _____ cm^3

Initial reading _____

Reading after trial No. 1 _____; Change in vol. _____ cm^3 Reading after trial No. 2 _____; Change in vol. _____ cm^3 Reading after trial No. 3 _____; Change in vol. _____ cm^3 Average ΔV _____ cm^3 Correction factor $CF = V_c/\Delta V =$ _____[Note, if correction factor is less than ± 0.002 , neglect it.]

COEFFICIENT OF PERMEABILITY (Constant Head, Falling Head)

Data Sheet 13

Project _____ Job No. _____

Location of Project _____

Description of Soil _____

Tested by _____ Date of Testing _____

Sample Dimensions: Diam. _____ cm; Area _____ cm²; Ht. _____ cm

Wt. soil + pan Init. _____ g

Vol. _____ cm³

Wt. soil + pan Final _____ g

Unit wt. _____ kN/m³

Wt. of Sample _____ g

Constant Head

Test data

Test data used

Test No.	t, s	Q, cm ³	T, °C	Test No.	t, s	Q, cm ³	T, °C
1							
2							
3							
4							
Average "							

$$k_T = QL/Aht = \underline{\hspace{2cm}}$$

$$\eta_T/\eta_{20} = \underline{\hspace{2cm}}$$

$$= \underline{\hspace{2cm}} \text{ cm/s}$$

$$k_{20} = k_T \eta_T/\eta_{20} = \underline{\hspace{2cm}} \text{ cm/s}$$

Falling Head

Standpipe = [burette, other (specify)] _____

Area of standpipe, a = _____ cm²

Test data^b

Test data used

Test no.	h ₁ , cm	h ₂ , cm	t, s	Q _{in} , cm ³	Q _{out} , cm ³	T, °C	Test no.	h ₁ , cm	h ₂ , cm	t, s	T, °C
1											
2											
3											
4											
Average											

$$\eta_T/\eta_{20} = \underline{\hspace{2cm}}$$

$$k_T = \frac{aL}{At} \ln h_1/h_2 = \underline{\hspace{2cm}} = \underline{\hspace{2cm}} \text{ cm/sec}$$

$$k_{20} = k_T \eta_T/\eta_{20} = \underline{\hspace{2cm}} = \underline{\hspace{2cm}} \text{ cm/sec}$$

^aUse averaged values only if there is a small difference in test temperature, say, 1–2°C.

^bThis test can be considerably simplified by using the same values of h₁ and h₂ each time, otherwise you cannot average these values regardless of T.

CONSOLIDATION TEST

Data Sheet 14

Project _____ Job No. _____

Location _____ Boring No. _____ Sample No. _____

Description of Soil _____ Depth of Sample _____

Tested By _____ Date of Testing _____

Consolidometer type _____ Ring no. _____

Multiplication ratio of load device _____

Ring dimensions: Diam. _____ Area, A _____ Ht. _____

Initial ht. of soil, H_i _____

Specific gravity of soil, G_s = _____

Wt. of ring + specimen
at beginning of test = _____

Wt. of ring = _____

Wt. of wet soil, W_t = _____

Computed dry weight
of soil, W_s' = _____

Oven dry wt. of soil,^a W_s = _____

Computed Ht. of solids^b H_s = _____

Initial Ht. of voids, $H_v = H_i - H_s$ = _____

Initial degree of saturation, $S_i = (W_t - W_s)/(H_v A)$ = _____

Initial void ratio $e_i = H_v/H_s$ = _____

Final Test Data (obtained at end of load testing)

Initial dial reading _____

Final dial reading _____

Change in sample ht. _____

Final ht. of voids, H_{vf} _____

Final void ratio, $e_f = H_{vf}/H_s$ _____

Water content determination

Wt. of can + wet soil = _____

Wt. of can + dry soil = _____

Wt. of can = _____

Wt. of water = _____

Wt. of dry soil = _____

Initial water content w_i = _____

Final water content determination

Final wet wt. + ring^c _____

Final dry wt. + ring _____

Oven dry wt. of soil, W_s _____

Final water content, w_f _____

Final degree of sat. S _____ %

^aObtained from Final Water-Content data.

^bUse either G_s or final water-content data for $S = 100\%$.

^cBe sure to include any soil extruded from ring which is in consolidometer.

CONSOLIDATION TEST (Time-compression data)

Data Sheet 15

Project _____ Job No. _____

Location of Project _____ Boring No. _____ Sample No. _____

Description of Soil _____ Depth of Sample _____

Tested By _____ Date of Testing _____

Loading Test Data

Load _____ kPa

Load _____ kPa

Date applied _____

Date applied _____

Applied by _____

Applied by _____

[illegible]

Clock time and date	Elapsed time, min	Dial readings \times _____	
		Original	Adjusted
	0		
	0.1		
	0.25		
	0.5		
	1		
	2		
	4		
	8		
	15		
	30		
	60		

*Insert gage subdivisions 0.01 mm/div, etc.

Data Sheet 16

Tested By _____ Date of Testing _____

Initial ht. of voids, H_v = _____ Initial void ratio, e_i = _____

[illegible]^eFrom the dial reading vs log *t* curves.

Data Sheet 17

Tested By _____ Date of Testing _____

Water content, $w\%$ _____ Dry unit wt. _____ LRC _____

[illegible]

Unconfined compressive strength $q_u =$ _____ Cohesion $= q_u/2 =$ _____

TRIAXIAL COMPRESSION TEST (Cohesive, Cohesionless)

Data Sheet 18

Project _____ Job No. _____
 Location _____ Boring No. _____ Sample No. _____
 Description of Soil _____ Depth of Sample _____
 Tested By _____ Date of Testing _____

Fill in the blanks with data and appropriate units.

Sample Data

Dimensions of test specimen: Diam. $D_0 =$ _____ Area $A_0 =$ _____ $L_0 =$ _____

Vol. $V_0 =$ _____ Water content $w =$ _____ % Degree of Saturation, $S =$ _____ %

Wt. $W_0 =$ _____

For Cohesionless soils

Initial wt. of container + sand = _____

Final wt. of container + sand = _____

Wt. of sand used in specimen, $W_o =$ _____

Specific gravity of sand $G_s =$ _____

Vol. of soil solids in test specimen $V_s =$ _____

Vol. of voids in test specimen (initial) $V_r =$ _____

Initial void ratio of test specimen $e_i =$ _____

Void ratio of sand at minimum density $e_{\max} =$ _____

Void ratio of sand at maximum density $e_{\min} =$ _____

Relative density of test specimen $D_r =$ _____

Unit wt. of test specimen (cohesive, cohesionless) $\gamma = W_0/V_0 =$ _____

Machine Data

Rate of loading _____ /min (insert centimeters or inches)

The following data may not be applicable if machine can be adjusted to tare these effects out of the load readings.

Cross section area of loading piston, $A_p =$ _____ cm^2

Upward load on piston $= A_p \sigma_3 =$ _____ kg

Wt. of loading piston $=$ _____ kg

Computed value of initial sample load $=$ _____ kg (If upward load is larger than weight of piston, ignore.)

Data Sheet 19

Tested By _____ Date of Testing _____

Sample Data: Area $A_0 =$ _____ Length $L_0 =$ _____

Machine Data: Load rate = _____ ./min Load ring constant LRC = _____ /div.

[illegible]

"The Deviator stress computation shown is based on taring the loading system so that the load ring reading is the deviator load value. $\sigma = (\text{Col. 3} \times \text{LRC})/A'$.

Maximum value of vertical stress $\sigma_1 = \sigma + \sigma_3 =$ _____

Project _____ Job No. _____ Location of Project _____

[illegible]

Depth of Sample _____
 Tested By _____
 Date of Testing _____

Area of specimen $A_0 =$ _____

Length of specimen $L_0 =$ _____

Confining Pressure $\sigma_3 =$ _____

Rate of loading = _____/min Load ring constant LRC = _____

Initial burette reading = _____ cm³ Initial sample void ratio, e_0 = _____ Vol. of solids, V_s = _____

[illegible]

Note: Insert units in column headings as necessary.
 *Based on taring system so load ring reading is the deviator load value.

Data for Mohr's Circle:

Maximum deviator stress = _____

Principal stress σ_1 = _____

Effective principal stress = _____

Data Sheet 21

Tested by _____ Date of Testing _____

Data to Obtain Sample Density if not an Undisturbed Sample

$$\gamma_{\text{dry}} = \frac{\text{mass of dry soil}}{\text{volume of soil}}$$

Normal stress σ_n

Loading rate = _____ /min Load ring constant = _____ /div.

[illegible]

^aFor square samples, may use corrected specimen area at failure as $A' = A_u - b \Delta H$ to compute σ_u and τ .

Data Sheet 22

Wt. wet soil _____ Wt. dry soil, W_s _____ γ_{wet} _____ γ_{dry} _____

[illegible]

Mold no.			
Surcharge, N			
Initial wt. wet soil + mold + base plate			
Final wt. wet soil + mold + base plate			
Wt. of mold + base plate			
Initial wt. of wet soil, W_i			
Wt. of water absorbed, W_w			
% water absorbed = W_w/W_s			

Note: Insert units in column headings as necessary.

Data Sheet 23

Tested by _____ Date of Testing _____

Penetration, mm	Mold no. _____ Surcharge _____		Mold no. _____ Surcharge _____		Mold no. _____ Surcharge _____	
	Piston load dial reading (_____)	Load, kPa	Piston load dial reading (_____)	Load, kPa	Piston load dial reading (_____)	Load, kPa
0.000						
0.5						
1.0						
1.5						
2.0						
2.5						
3.0						
4.0						
5.0						
Final water content, $w\%$ (soaked, unsoaked) sample	Top $\frac{1}{3}$					
	Middle $\frac{1}{3}$					
	Bottom $\frac{1}{3}$					
	Average					

Note: Insert units in column headings as necessary.

VOLUMETRIC GRAVIMETRIC RELATIONSHIPS

Data Sheet 24

Project _____

Location of Project _____

Description of Soil _____

Tested by _____ Date _____

Volume of container, $V_c =$ _____ cm^3

Weight of dry soil + container = _____ g

Weight of soil + water + container = _____ g

Weight of container = _____ g

Weight of dry soil, $W_s =$ _____ gWeight of saturated soil, $W_t =$ _____ gVolume of water to saturate soil, $V_v =$ _____ ml

Compute:

$$V_s = V_c - V_v = \text{_____} = \text{_____} = \text{_____} \text{ cm}^3$$

$$\gamma_d = \text{_____} = \text{_____} \text{ g/cm}^3 \times 9.807 = \text{_____} \text{ kN/m}^3$$

$$\gamma_{\text{sat}} = \text{_____} = \text{_____} \text{ g/cm}^3 \times 9.807 = \text{_____} \text{ kN/m}^3$$

$$e = \text{_____} = \text{_____}$$

$$G_s = \text{_____} = \text{_____}$$

UNIT WEIGHT OF A COHESIVE SOIL

Data Sheet 25

Project _____ Job No. _____

Location of Project _____

Description of Soil _____

Tested by _____ Date _____

Boring No. _____ Boring No. _____

Depth _____ Depth _____

Wt. of sample, W_t = _____ g

Wt. of sample, W_t = _____ g

Vol. of container, V_c = _____ cm^3

Vol. of container, V_c = _____ cm^3

Initial grad. read. = _____ ml

Initial grad. read. = _____ ml

Final grad. read. = _____ ml

Final grad. read. = _____ ml

Volume of water, V_w = _____ ml

Volume of water, V_w = _____ ml

Volume of soil, V_s = _____ cm^3

Volume of soil, V_s = _____ cm^3

Wet density:

Wet density:

$$\gamma_{\text{wet}} = W_t/V_s = \text{_____ g/cm}^3$$

$$\gamma_{\text{wet}} = W_t/V_s = \text{_____ g/cm}^3$$

$$\gamma_{\text{wet}} = \text{g/cm}^3 \times 9.807 = \text{_____ kN/m}^3$$

$$\gamma_{\text{wet}} = \text{gm/cm}^3 \times 9.807 = \text{_____ kN/m}^3$$

Computation of Dry Unit Weight

Water content, $w\%$ = _____

Water content, $w\%$ = _____

$$\gamma_{\text{dry}} = \frac{\gamma_{\text{wet}}}{1 + w} = \text{_____ kN/m}^3$$

$$\gamma_{\text{dry}} = \frac{\gamma_{\text{wet}}}{1 + w} = \text{_____ kN/m}^3$$

$$\gamma_{\text{dry}} = W_s/V_s = \text{_____ kN/m}^3$$

$$\gamma_{\text{dry}} = W_s/V_s = \text{_____ kN/m}^3$$

